END PLATE CONNECTIONS
AND THEIR INFLUENCE ON
STEEL AND COMPOSITE STRUCTURES

by

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To my brother

Declaration

Except where specific reference has been made to the work of others, this thesis is the result of my own work. No part of this thesis has been submitted to any university or other educational establishment for a degree, diploma or other qualification.

Acknowledgements

The work described in this thesis was part of a research into the behaviour of semi-rigid connections in steel and composite structures. The work was conducted at the Engineering Department of University of Warwick over the period October 1989 to September 1992.

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SUMMARY

The structural members of steel frames are jointed by connections. These connections are neither rigid nor pinned but semi-rigid. The performance of steel structures is highly influenced by the behaviour of connections which is characterized by the moment-rotation curve. This thesis studies the effects of semi-rigid connections on steel and composite steel-concrete frames, in particular the behaviour and influence of end plate connections.

The first part of the thesis concerns the performance of unbraced planar steel frames with semi-rigid joints. Several aspects are investigated, concerned mainly with the serviceability limit state. Using the definition given in Eurocode 3 for a rigid connection, it is suggested that the conventional limit for the sway angle should be relaxed by 10% when the rotational behaviour of the joints is included in the analysis. For frames designed by the wind-moment method, it is proposed that deflections based on the assumption of rigid joints should be increased by 50% to allow for the connection flexibility. An approximate method, in which the stiffness of beams are reduced to account for joint flexibility, was found to be sufficiently accurate if deflections based on semi-rigid behaviour were to be calculated. Finally, studies on the ultimate limit state show that the resistance of a joint has significant effect on the collapse load of a frame, compared to the more modest influence of joint flexibility.

The second part of the thesis concerns the behaviour of composite connections in braced frames. This part consists of a concise collection of the available experimental data, a description of the test programme conducted by the author, a proposed method for prediction of connection stiffness and studies on redistribution of moments in composite beams. Eleven tests have been carried out on bare steel and composite end plate joints. Their moment-rotation behaviour is recorded and the influence of variables on the joint stiffness is pointed out. These variables are the amount of reinforcement in the concrete slab, the type of steelwork connection and the beam depth. Increase in the amount of reinforcement increases significantly the moment resistance of the composite joint but does not influence its initial stiffness. 1% reinforcement with respect to the area of concrete slab is proposed to be used for an efficient design. The increase in the depth of steel section increases the moment resistance and stiffness of composite connection but reduces its rotation capacity. The effect of semi-rigid composite connections on column stability is also studied and a value of 0.75 is proposed as the effective length factor for columns. The proposed method for prediction of connection behaviour is shown to be in satisfactory agreement with the test results. From the final studies in Part Two formulae are proposed for calculation of the required rotation capacity of composite connections.

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PART TWO

END PLATE CONNECTIONS IN BRACED COMPOSITE FRAMES

Notation

A_c	area of concrete slab (above decking)
A_r	cross sectional area of reinforcement
A_s	cross sectional area of steel section
B	breadth of the flange of steel section
C	secant stiffness of joint
C_i	constants
C_s	stiffness of steelwork connection
D	beam depth
D_b	distance between the centroid of top row of bolts and beneath the bottom flange of beam
D_{b1}	distance from the centroid of top row of bolts between the beam flanges to beneath the bottom flange of beam
D_{b2}	distance between the centroid of top row of bolts in the extended part of end plate and beneath the bottom flange of beam
D_c	depth of the column section
D_r	distance between the centroid of reinforcement and beneath the bottom flange of beam
D'	distance between the top face of the ribs of decking and beneath the beam bottom flange
E	modulus of elasticity
E_c	modulus of elasticity of concrete
E_r	modulus of elasticity of reinforcement
E_s	modulus of elasticity of structural steel
F	force
F_b	force in the top row of bolts in the mechanical model
F_i	force in component i
$F_{1.Rd}$	force in the first row of bolts below tension flange due to moment M_{Rd}
F_r	force in reinforcement in the mechanical model

I	second moment of area
I_b	second moment of area of beam section
I_c	second moment of area of column section
I_s	second moment of area of beam in the substitute frame
K	member stiffness
K_a	stiffness of compression spring in the mechanical model
K_b	beam stiffness
K_b	stiffness of the spring representing the stiffness of steelwork joint in the mechanical model
K_{bb}	stiffness of bottom beam in substitute frame
K_{bt}	stiffness of top beam in substitute frame
K_c	column stiffness in substitute frame
K_c	stiffness of the spring representing the concrete slab in the mechanical model
K_p	stiffness of the spring representing the profiled steel sheeting in the mechanical model
K_r	"relative stiffness" of beam
K_r	stiffness of the spring representing reinforcement in the mechanical model
$\overline{K_r}$	ratio of "relative beam" stiffness to the actual beam stiffness
$K_{r,c}$	stiffness of the spring representing the reinforcement and concrete in the mechanical model
K_s	stiffness of the spring representing shear connectors in the mechanical model
L	length of structural member
L_b	length of beam
M	bending moment
M_c	resistance moment of composite connection
$M_{crac.}$	test moment at the time of first crack
M_e	elastic moment at the support of a fixed-end beam
M_h	hogging moment at support
M_j	moment at the end joint of composite beam
M_{ji}	joint moment at construction stage

$M_{Pl.Rd}$	plastic moment resistance of beam (hogging)
M_{pc}	plastic design moment resistance of composite beam in hogging bending
M_{ph}	plastic moment resistance of composite beam in hogging bending
M_{ps}	plastic moment resistance of composite beam in sagging bending
M_{pss}	plastic design moment resistance of steel section
M_{Rd}	design resistance moment of connection
M_{sc}	resistance moment of steelwork connection
M_{si}	sagging moment of steel beam at construction stage
M_u	ultimate moment
M_y	yield moment
M_{ys}	sagging yield moment of composite beam
P	force
P_k	characteristic resistance of a stud connector
R_b	force in the top row of bolts
R_{b1}	force in the top row of bolts between the beam flanges
R_{b2}	force in the top row of bolts in the extended part of end plate
R_f	force in the bottom flange of beam
R_r	force in reinforcement
S	cladding stiffness
T	beam flange thickness
Z	rotation of joint for a unit value of moment
b_e	effective breadth of concrete slab
d	distance between two extreme end rows of bolts
f	column flange thickness
f_{cu}	cube strength of concrete
f_y	yield strength of structural steel
f_{yr}	yield strength of reinforcement

f_{ys}	yield strength of steel section
h	column height
h_1	distance from first row of bolts below tension flange to centre of compression
k	short term stiffness of a stud connector
k	ratio of support moment resistance to midspan moment resistance of composite beam
k_i	stiffness factor for component i
l	effective length of column
l_y	yielded length of composite beam in sagging bending
$\overline{m}$	non-dimensionalised moment
r	redistribution ratio
s	spacing of shear connectors
s	shape factor
s_m	modification multiplier for beam span
t, t_e	end plate thickness
t_{fc}	column flange thickness
t_{wc}	column web thickness
w	uniformly distributed load
w_s	serviceability load
w_u	ultimate state load
x	depth of the beam web in compression
x_p	distance between the plastic neutral axes of composite and steel section
Δ	displacement
Δ_b	displacement at the top row of bolts in the mechanical model
Δ_r	displacement at the reinforcement in the mechanical model
Δ_s	displacement at the steel-concrete interface in the mechanical model
Φ	total end rotation of a beam connected to a column
Φ_o	end rotation of beam associated with yielding at midspan

$\bar{\Phi}$	non-dimensionalised sway
α	load factor
α_e	modular ratio of steel to concrete
β	multiplier
δ	displacement, deflection
ε	strain
η	"distribution factor"
γ	interface slip
γ_m	partial safety factor
γ_p	plastic slip of a stud connector
κ	curvature of composite beam
μ_i	modification factor for component i
ω	multiplier to "semi-rigid force ratio"
ϕ	connection rotation
$\bar{\phi}$	non-dimensionalised rotation
ϕ_d	rotation capacity of connection
ϕ_i	initial rotation of steel beam end at construction stage
ϕ_s	rotation of steelwork connection
ρ_f	"semi-rigid force ratio"
ρ_r	"semi-rigid force ratio" taking account of position of plastic neutral axis and beam depth
σ_y	yield stress of steel section
θ	rotation of a structural member
θ_b	rotation at the bottom of column
θ_e	elastic rotation of composite beam
θ_p	plastic rotation of composite beam
θ_t	rotation at the top of column

Chapter 1
INTRODUCTION

The steel skeleton has become the most popular type of structural system in modern construction for buildings. The skeleton consists of members as beams and columns jointed together through connections. The possibility of fabricating these structural elements in the workshop under good quality control, plus the fact that erection of the steel skeleton is less dependent on environmental conditions than other forms of construction, have contributed to this popularity.

Following the erection of steel skeleton, further economy will be achieved if the floors act as structural elements in conjunction with the steel beams, as composite members. This results in smaller member sections, accelerates the execution process and facilitates the construction works. The use of fabricated decking adds to the attractiveness of composite construction because it provides a platform for the labour and formwork for concrete. Further benefit may be gained if the columns and beams are encased in concrete which will also provide some degree of protection against fire.

Although a structure behaves three-dimensionally, for analysis, it is usually idealised to a set of planar sub-structures or frames. The planar frames of each direction are then analysed and their interactive effects are superimposed where appropriate. Although sophisticated analysis software is available for three-dimensional structures, the analysis of individual planar frames is still common in design offices. The studies reported in this thesis are applicable to frames idealised in this way.

This chapter describes different categories of plane frame, including a classification based on the type of beam-to-column connection expected to be used. Methods of analysis and design are briefly described, sufficient to demonstrate the scope for the research presented in later chapters. The contents of these are given in outline.

1-1-Principal Definitions

A structure is designed to resist the applied actions such that both serviceability and ultimate limit states are satisfied. The ultimate limit states are those associated with collapse of the structure, while serviceability limit states correspond to the states beyond which specified service criteria are no longer met. The individual elements of a structure must also be checked to resist the design actions.

1-1-1-Resistance to Sway

All frames should have sufficient stiffness to limit lateral sway. The resistance to sway may be provided by either a bracing system or the stiffness of the connected beams and columns within the structure.

According to Eurocode 3(1992), frames may be classified as sway or non-sway, and as braced or unbraced:

> "A frame may be classified as non-sway if its response to in-plane horizontal forces is sufficiently stiff to be acceptably accurate to neglect any additional internal forces or moments arising from horizontal displacements of its nodes."
>
> "A frame may be classified as braced if its sway resistance is supplied by a bracing system with a response to in-plane horizontal loads which is sufficiently stiff for it to be acceptably accurate to assume that all horizontal loads are resisted by the bracing system."
>
> According to EC3, a braced frame may be treated as fully-supported laterally.

1-1-2-Second-Order Effects

The influence of the axial force on the flexural stiffness of a member is usually termed the $P-\delta$ effect. The influence of the vertical loads on the sidesway stiffness of a column or frame is referred to as the $P-\Delta$ effects. These effects are shown in Fig. 1-1 and generally are referred to as the second-order effects. It is evident that the $P-\Delta$ effects have greater influence on the overall performance of the frame, and they are of particular significance in sway frames.

1-1-3-Classification of Sections

The moment resistance of steel and composite sections is limited by the susceptibility of the section to local buckling. The applicability of plastic stress block analysis is dependent on the classification of the section:

a) Class 1 Plastic; a plastic hinge can develop with sufficient rotation capacity required for plastic analysis.

b) Class 2 Compact; the full plastic moment can develop but local buckling may prevent development of a plastic hinge with sufficient rotation capacity required for plastic analysis.

c) Class 3 Semi-Compact; the stress at the extreme fibre can reach the design strength but local buckling may prevent the development of full plastic moment.

d) Class 4 Slender; local buckling may prevent the stress from reaching the design strength.

The rotational behaviour of the sections are shown in Fig. 1-2. In the codes of practice for steel and composite construction, explicit allowances have been made for the use of non-plastic sections.

1-1-4-Simple and Continuous Construction

In the design of steel frames it is customary to assume the joints are either pinned or rigid. In a frame designed as pinned, no moments transfer between beams and columns, which results in a conservative choice of beam sections. On the other hand, in a frame with rigid joints complete rotational continuity exists at connections and the columns are designed for the resulting moments transmitted. The different assumptions are illustrated in the beam of Fig. 1-3 where the end moments change due to the flexibility of connections.

The two ideal design approaches are respectively known as "simple" and "continuous" construction. In reality, the actual behaviour of connections in frames is somewhat between these two extremes. Extensive experimental investigations show that joints in practice behave over a wide spectrum with moment transfer characteristics varying from those for a very flexible connection to those for a relatively stiff joint.

The relative complexity of the necessary design calculations and lack of comprehensive information on the performance of the full range of connections has resulted in persistent use of the two extreme idealisations. Although substantial work has been done on the behaviour of steel connections, there is still demand for research to provide the practical data required in design. Composite connections suffer particularly from shortage of information to enable designers to include their effects when analysing the structure.

1-1-5-Semi-Rigid Action

The moment distribution in a frame depends on the moment-rotation behaviour of the connections. Typical moment-rotation curves for the most commonly used beam-to-column connections are shown in Fig. 1-4. In this figure, the vertical axis (M) represents the perfectly rigid connection, and the horizontal axis (ϕ) reperesents the perfectly pinned connection. As shown, the behaviour of most real connections lies between the two axes.

The connections with $M-\phi$ curves closer to the horizontal axis are flexible, whilst those closer to the vertical axis are stiff. As shown in Fig. 1-4, the rotational characteristics of joints differ from one type of connection to another.

It is seen that the connections used in practice are neither pinned nor rigid, rather they possess some degree of rotational resistant. The term "semi-rigid" denotes such connections. As a result, the more correct approach is to classify all steel frames as "semi-rigid" with "simple" and "continuous" construction being extreme idealisations.

In order to demonstrate the difference between "rigid" and "semi-rigid" connections, reference should be made to Fig. 1-5. The connection in Fig. 1-5(a) is rigid. It is seen that the centreline of the beam remains perpendicular to that of the column after deformation under loading. The overall rotation of the beam-column assembly is θ whilst no rotation occurs between the beam and the column.

The connection shown in Fig. 1-5(b) is semi-rigid. The column rotates through an angle θ while the beam rotation is Φ. The relative rotation ϕ is defined as the rotation of the connection:

$$\phi = \Phi - \theta \tag{1.1}$$

The moment-rotation relationship of a semi-rigid connection is characterised by the following parameters (see Fig. 1-6):

1) The moment resistance, which is equal to the peak value of the moment-rotation curve.

2) The design moment, which is the value of moment to be resisted by the connection to fulfill the design requirements of the joint.

3) The rotation capacity, which is the maximum rotation achieved at the design moment resistance of the connection.

4) The rotational stiffness, shown in Fig. 1-6, which can be the initial stiffness, the secant stiffness, or the tangent stiffness at any point to the $M-\phi$ curve.

1-1-6-Classification of Connections

For design considerations, beam-to-column connections may be classified by their stiffness as:

1) Nominally pinned; no moment is transmitted between the beam and the column. Thus the connection is only capable of transmitting vertical shear and axial force.

2) Fully rigid; no relative rotation occurs between the beam and the column and the corresponding moments are transmitted between these members.

3) Semi-rigid; a certain degree of restraint is provided between the beam and the column. The connection rotates relative to the adjacent members and transmits a moment dependent on its stiffness.

In addition to classification by stiffness, the beam-to-column connections may be classified by resistance, with respect to the design moment resistance of the connected beam. This classification divides the connections into:

1) Nominally pinned; the connection should be able to transmit the forces calculated in design, without developing significant moments which might adversely affect members of the structure.

2) Full-strength; the design resistance of the connection should be at least equal to that of the beam.

3) Partial-strength; the design resistance of the connection is less than that of the beam.

Thus, a connection can be assumed rigid or semi-rigid and at the same time be either partial- or full-strength. A connection which can develop a moment up to the resistance of the adjacent beam provides maximum strength for the frame. However, the fabrication of such connections is costly, partly because they usually need stiffeners. Therefore tendency in the construction industry has been to use nominally pinned connections. There is also increasing interest in partial strength connections, which possess some degree of stiffness.

1-2-Design and Analysis of Steel Frames

1-2-1-Methods of Frame Design

The traditional pinned assumption for joints results in the design of braced frames in which the bracing is to provide lateral stability. On the other hand, when the frame is assumed perfectly rigid, this function is given to the moment resistant connections, capable of transmitting the internal moments. The question of choosing the design method depends partly on cost, comparing bracing with the expenditure on labour and material being used to stiffen the connections. However, the design will not be exact and not necessarily economic, regardless of the choice of braced or unbraced frame, if the semi-rigid behaviour is not taken into account. Extensive investigations by the Steel Structures Research Committee(1931)(1934)(1936) and Johnston & Hetchman(1940) have shown that a benefit could be achieved in economy by semi-rigid design compared to assuming simple connections.

Recent design codes recognize analysis and design of structures with semi-rigid joints for both sway and no sway cases. In addition, a well-established method known as the "wind-moment" approach is extensively used in design; this employs pinned and rigid joints in sequence, as explained later.

1-2-1-1-American Standard, AISC

Current US practice is summarized in the Manual of Steel Construction(1986). Part 6 of this manual permits two basic "types of construction" as follows:

a) Type FR (fully restrained), assumes that the beam-to-column connections have sufficient rigidity to hold the original angles between intersecting members virtually unchanged.

b) Type PR (partially restrained), assumes that the connections of beams and girders possess insufficient rigidity to hold the original angles between intersecting members virtually unchanged.

The use of Type PR construction depends on the evidence of a predictable proportion of full end restraint. Where the connection restraint is ignored, "simple framing" construction can be employed. This type assumes that under gravity loads the ends of the beams and girders are connected for shear only and are free to rotate. In this case, the connections must have sufficient inelastic rotation capacity to avoid overload of fasteners or welds under combined gravity and lateral loading. As will be seen later, "simple" design of AISC treats the frame by an approach known in Britain as the "wind-moment" method.

Although the AISC Manual recognizes the semi-rigid nature of joints, it gives no guidance for incorporating the behaviour of connections into frame design. Also the partial restraint provided by semi-rigid joints is not accounted for in the Manual. However, this code calls for the second-order effects to be considered for both braced and unbraced frames.

1-2-1-2-British Standard , BS5950

In BS5950:Pt 1(1990), methods of design are classified as:

a) Simple design, in which the connections are assumed not to develop moments adversely affecting either the members or the structure as a whole. The distribution of forces is determined on the basis of pinned joints, which may result in some non-elastic deformation of material.

b) Rigid design, in which "the connections are assumed to be capable of developing the strength and/or stiffness required by an analysis assuming full continuity".

c) Semi-rigid design, in which some degree of connection stiffness is assumed. In this method two alternatives are proposed. The first uses experimental data including connection moment-rotation behaviour. The second is to restrain the end of the beams in "simple" design by an end restraint moment up to 10% of the free moment applied to the beam. For the latter recommendation the welds and fasteners are designed for the actual resistance of other parts of the connection, not the assumed (up to 10%) moment, to avoid brittle forms of failure.

The 10% of the free moment supposed to be absorbed by the joint has been based on the mimimum moment resisted by a flexible joint. However, even unstiffened flange cleat connections can resist more than this (Davison(1987)). A better approach is to account for each type of connection individually by adopting simple criteria. In the absence of such guidance, the above percentage can be modified to a greater value where a semi-rigid connection is utilized. Benterkia(1991) suggested a minimum of 20% end restraint based on the experimental results from other researchers and his numerical study on semi-rigid frames.

1-2-1-3-European Unified Standard, Eurocode 3

In Part 1.1 of EC3(1992), the term "type of framing" is used to distinguish between frames which are either:

a) Simple, in which the members are assumed to be effectively pin connected and the connections do not develop moment. Thus, the structure is statically determinate.

b) Continuous, in which full continuity is assumed. Elastic, rigid-plastic and elastic-plastic analyses may be used with appropriate assumptions. Rigid; full-strength; and rigid full-strength connections shall be used in these analyses respectively.

c) Semi-continuous, in which the characteristics of connections are taken into account. Elastic, rigid-plastic and elastic-plastic analyses may be used. Elastic analysis should be based on reliably predicted design moment-rotation or force-displacement characteristics of the connections. Rigid-plastic analysis should be based on the design moment resistances of the connections which have been demonstrated to have sufficient rotation capacity. Elastic-plastic analysis should be based on the design moment-rotation characteristics of the connections.

Although EC3 gives guidance on categorizing a frame with regard to the performance of the connections, it is still lacking information on different types of joints. The only type of connection which has been dealt with in EC3 is the end plate connection (flush and extended). Even for this type of joint, the complexity of calculation procedures given in Annex J of the code may discourage the designer from making use of the method.

1-2-1-4-Wind-Moment Method

For unbraced steel frames, an established technique is to rely on the rotational stiffness of the connections to provide resistance to wind, even though such restraint is ignored under the action of gravity loads. This approach is termed the "wind-moment" or "wind-connection" method. The method has been explained and evaluated by Anderson et al(1991)(1992). The following has been extracted from these references.

The method in its usual form assumes:

1) under gravity load, the connections act as pins (Fig. 1-7(a));
2) under wind load, the connections behave as rigid joints, with points of contraflexure at the mid-height of columns and mid-length of beams (Fig. 1-7(b)).

Members and connections are proportioned initially to withstand gravity load. The internal forces and moments due to gravity load and wind (Fig. 1-8(a) and (b)) are then combined in appropriate load cases. The design for strength is completed by amending the initial section sizes and other details for the members and connections, to withstand the combined effects.

No calculation is made due to $P-\Delta$ effects. It is assumed that these can be accounted for by using effective column lengths greater than the true lengths, for axes about which sway can occur.

For serviceability, sway deflections are calculated assuming connections are rigid.

The advantage of the method is simplicity. As the frame is rendered statically determinate, internal moments and forces are not dependent on the relative stiffnesses of the members. The need to repeat the analysis to correspond to changed section sizes is thereby avoided. Consequently, the method has been used extensively, although it has not been verified as a generally-applicable approach.

This method has been used in Part One of this thesis to design a number of frames to be analysed with semi-rigid joints.

Analytical justifications, including the effect of connection flexibility, have been carried out on the method and the results have been summarized by Anderson et al(1991)(1992), based on the fact that buildings designed according to this method have proved satisfactory in use.

1-2-2-Methods of Frame Analysis

A frame can be analysed by any rational method in which the equilibrium and compatibility conditions are satisfied. This necessitates that the mechanical behaviour of the materials of the frame's components are known, and the effects of non-linearities are recognised. The non-linearities in a semi-rigid frame may arise from the following:

1) The non-linear $M-\phi$ relationship of the connections.

2) The geometrical non-linearity of the members referred to as the $P-\delta$ and $P-\Delta$ effects.

3) The material non-linearities which are inherent to the mechanical properties of materials.

Two theories may be applied in analysis, depending on whether or not the $P-\Delta$ effects are taken into account. According to EC3, a frame may generally be analysed by either:

1) First-order theory using the initial geometry of structure; hence it is employed in braced or non-sway frames.

2) Second-order theory taking into account the influence of deformations of the structure; thus it can be used for all types of frames.

Based on the types of non-linearities and the degree of accuracy required, three types of analysis may be used:

a) Linear elastic analysis, in which the linear behaviour of materials and connections are assumed. No account is taken of the $P-\Delta$ effects; hence it is acceptable only for stiff connections and low values of displacement.

b) Non-linear elastic analysis, in which non-linear behaviour of materials and connections are used as well as the geometrical non-linearities of the framed structure.

c) Inelastic analysis, in which the yielding of the members is considered and the above non-linearities are taken into account. The global analysis of frame may be performed on the basis of rigid-plastic or elastic-plastic theory.

In Part One of this thesis, reference will be made to the above theories where analysis of frames is described.

1-2-2-1-Global Analysis of Semi-Rigid Frames

The global analysis of frame is carried out with different combination of loading. Using the simplified rules for combination of actions given in EC3, the following load cases are considered for the design and analysis of the framework:

1) Dead, imposed and wind loading.

2) Dead and wind loading.

3) Dead and imposed loading.

Appropriate partial safety factors for loads are proposed in the codes of practice to be applied to the loads at serviceability and ultimate limit states.

Global analysis under service loading is required to determine deflections. The second-order effects in this state are less significant than for the ultimate limit states and it is common practice in the design of continuous framing for such effects to be ignored. For semi-continuous framing, second-order effects need considering and an appropriate method can be chosen from those explained above. The joint behaviour may be taken as linear elastic, provided that design at ultimate limit state is consistent with elastic global analysis (ECCS(1992)). Plastic global analysis should be used at SLS when elastic analysis has been found to be inappropriate because the moment resistance of a connection or a member has been attained. This matter will be discussed in Chapter 3 and a simplified method based on an effective beam model will be presented.

Although design based on plastic analysis promises greater economy than use of elastic methods, the design of unbraced frames have been found to be mostly governed by deflection limits, and threfore elastic methods are more appropriate for global analysis at ULS (ECCS(1992)). However, not all semi-continuous frames will be governed by serviceability, particularly where heavy vertical loading is combined with low wind loading (Reading(1989)). Plastic global analysis is then more appropriate to achieve economy. Irrespective of whether an elastic or plastic analysis is used, EC3 permits first-order analysis provided the frame is classified as "non-sway".

1-2-2-2-Beam Line Method

The beam line method may be used to analyse a beam with semi-rigid connections. Although this approach can be extended to a semi-rigid frame, particularly by use of subframes (Benterkia(1991)), the concept of the method will be explained for a beam with two semi-rigid connections at its ends. This method will be used in Part Two of this thesis for validation of a computer program.

Fig. 1-9 illustrates the concept of the beam line method. The method was proposed by Batho & Rowan in the reports to the Steel Structures Research Committee (1931)(1934)(1936).

The end rotation Φ of a beam of span L, with flexural stiffness EI, equal end moments M and under uniformly distributed load w is given by:

$$\Phi = \frac{wL^3}{24EI} - \frac{ML}{2EI} \tag{1.2}$$

For a fixed ended beam, the end rotation is zero and from the above equation, the corresponding end moment is $\frac{wL^2}{12}$. For a simply supported beam, the end moment is zero and the corresponding rotation is $\frac{wL^3}{24EI}$. For a given value of w, the two extremes form the "beam line" as shown in Fig. 1-9.

Also shown in Fig. 1-9 is the moment-rotation curve of a typical connection. The beam line and the moment-rotation curve intersect at a point, where the corresponding values of M and Φ represent the end restraint conditions of a beam with such a connection at its ends.

As w increases, the beam line moves further from the centre of coordinates. For a fixed ended beam, yielding occurs at the support at the end moment equal to M_y. For a pinned ended beam, when yielding occurs at the midspan, the end rotation is Φ_o. These situations are shown in Fig. 1-9 as the "yield beam line" which was developed by Sommer(1969).

If plasticity develops at the support under the moment M_y, this will coincide with the yielding at midspan at a rotation equal to $\frac{\Phi_o}{2}$ (Nethercot(1985b)). This is shown in Fig. 1-9 as the "two-phase beam line".

1-3-Scope of Part One

Part One of the thesis deals with different aspects of semi-rigid action in unbraced steel frames.

To decide on the appropriate methods for design and analysis of a frame, the behaviour of joints within the frame has to be identified. Once the joint behaviour is recognized, the frame can be taken to be either rigid, semi-rigid or flexible. The assumption of a rigid or a flexible frame is made however for simplicity, to avoid the less convenient methods of accounting for semi-rigid action. If connections within the frame cannot be assumed as either rigid or flexible, because of their moment transfer characteristics, then the proper solution is to design and analyse the frame semi-rigidly.

On the basis of the above argument, EC3 has adopted classification criteria for connections in braced and unbraced frames. These will be explained in Part One. The boundary defined between rigid and semi-rigid joints in unbraced steel frames will be evaluated to determine the influence of such classification on frame behaviour.

The use of a classification criterion and consequently the adoption of suitable design and analysis methods necessitates the knowledge of connection behaviour. This can be gained by experiment on the individual joints in a frame; or more practically, by prediction of the connection behaviour, based on an appraisal of the mechanical behaviour of its components.

Many attempts have been made by researchers to predict connection behaviour. Recently, EC3 has given a method for stiffness and resistance of end plate connections. The prediction equation recommended by EC3 is examined in Part One against a previously proposed equation (Frye & Morris(1975)). The influence of the prediction method on sway deflections is examined in the context of frames designed by the wind-moment method.

The sway response of semi-rigid frames can be obtained by the methods of analysis described earlier. The methods account for the behaviour of the joints and perform a global analysis to find the displacements. Simplified methods are also available for sway calculation. A simplified method based on an effective beam model is explained in Part One and is examined against results from rigorous analysis.

The behaviour of frames at collapse is highly dependent on the performance of semi-rigid connections within the frame. The effect of semi-rigid partial- and full-strength connections on a sample frame is highlighted by the results of plastic analysis of the frame. This study is the closing subject in Part One.

The background necessary for the above studies is given in Chapter 2 while the studies themselves are reported in Chapter 3. In Chapter 2 an overview of experimental work on steel connections is first made with particular attention to end plate connections. The attempts to derive prediction equations for the joint behaviour based on these experiments are elaborated. This is of significance not only for the work described in Chapter 3 but also for the approach to a prediction equation for composite joints pursued in Part Two of this thesis. Methods of including the moment-rotation characteristic of connection in the frame analysis are also described in Chapter 2 and indications of frame response to semi-rigid action of joints are given. Finally the background to the effect of joint behaviour on the stability of columns is summarized.

1-4-Design and Analysis of Composite Frames

A composite frame is defined in EC4(1992) as "... a framed structure ..., in which some or all of the beams and columns are composite members and most of the remaining members are structural steel members". A composite member is "a structural member with components of concrete and of structural or cold-framed steel interconnected by shear connection...". The terms defined in section 1-1 are general and can be applied to both steel and composite construction.

In composite construction, two phases occur. The first phase is the erection of steelwork and concreting of the floors. In this phase, the frame is subjected to construction loads. The second phase is after completion of constructional work, and the building is subjected to imposed loads from use as well as the dead loads. In the construction stage, the beams may be propped or unpropped, which has a significant effect on the stress state of the steel beam both at this stage and subsequently.

To develop the plastic resistance moment of the composite section, it is necessary to transfer a longitudinal force across the steel-concrete interface equal to the lesser of the resistance of the concrete flange and the resistance of the structural steel section. Full shear connection is provided where the total resistance of the connectors between the point of maximum positive moment and each end support is not less than the design longitudinal force. Where the maximum positive moment is less than the plastic resistance moment of the composite section, the actual number of connectors may be reduced below the number for full connection. This is known as "partial shear connection" which reduces the bending resistance of the beam.

The analysis of a composite beam section requires an effective breadth to be taken for the concrete slab. The effective breadth in design for the sagging region is different to that for the hogging region. This is due to the change of stress distribution in the composite section, concrete being subjected to compressive stress in sagging bending and tensile stress in hogging bending.

"Cracked" and "uncracked" (or "gross") sections may be used for calculating the second moment of area of the composite section in hogging and sagging regions. In the former the tensile strength of concrete is neglected but the effectively anchored reinforcement within the effective breadth is employed. In the latter the concrete within the effective breadth is included, but the reinforcement is neglected (see Fig. 1-10). The detailed rules given in the codes of practice for global analysis of the composite beams account for the use of cracked or uncracked sections.

1-4-1-Codes of Practice

The codes of practice have extended the definitions of the types of framing for the design of steel frames to composite construction. They have also adopted loading combinations similar to those for steel frames. Account has to be taken though of the staged construction of composite beams and slabs.

For the studies reported in Part Two of this thesis, the British Standard and the Eurocode specifications need be considered.

Both codes describe "cracked" and "uncracked" analyses for composite beams. For the cracked section method, they take the flexural stiffness as the cracked value over a length of 15% of the span on each side of an internal support, and as uncracked values elsewhere. For the uncracked section method, the flexural stiffness is taken as the uncracked value throughout the beam's length. Both codes permit partial shear connection. They also assume the same proportions for the effective breadths.

1-4-1-1-British Standard, BS5950:Pt 3

BS5950:Pt 3.1 makes a distinction between the global analysis by which the moments and forces in the structure are determined; and the procedures for member design. In the former, elastic global analysis may be used without any restriction but plastic global analysis should only be used in structures where the members satisfy certain criteria given in the code. The plastic moment resistance is used for sections which are Class 1 Plastic or Class 2 Compact, otherwise the elastic moment resistance of the section is used.

In BS5950:Pt 3.1 no mention is made of the connections between beams and columns, and hence no account is taken of their influence on composite frames.

1-4-1-2-European Unified Standard, Eurocode 4

The two classification systems for steel frames employed in EC3 have also been adopted for composite frames. One concerns resistance to sway, and the numerical criteria given in EC3 to classify a structure as braced or unbraced have been used in EC4. Because of the rarity of sway frames in practice, composite sway frames are not treated at present in Part 1 of EC4.

The second classification system in EC3 relates the method of global analysis to the behaviour of connections and is also used in EC4. This classification divides frames into:

a) Simple frames: Statically determinate structures with nominally pinned connections.

b) Continuous frames: Elastically analysed structures in which connections are either nominally pinned or rigid. Alternatively, rigid-plastic analysis can be used in which either full strength or nominally pinned connections are assumed.

c) Semi-continuous frames: Rigid-plastic analysis is used with partial or full strength or nominally pinned connections.

Elastic analysis of semi-continuous frames is excluded from EC4 because at present there is no generally accepted method to predict the stiffness of the connections. Rigid-plastic analysis, with connections classified on the basis of their resistance, can be used in such frames. In order to use elastic-plastic analysis for semi-continuous frames, it is again necessary to know the stiffness of the connections.

In EC4 reference has been made to Chapter 6 of EC3 where the connections are classified. The restricted scope of the rules in EC3 has already been stated. Although the procedures of Annex J of this code for end plate joints can be used in the construction stage, or in connections assumed unreinforced, there are no rules in either EC3 or EC4 for joints acting compositely. In EC4 the main advice to the designer is a general remark to take account of the forces in the reinforcement and concrete. A further comment is also made concerning the benefit of improvement in the buckling resistance of the column web due to encasement in reinforced concrete.

1-4-2-Need for Research on Composite Frames with Semi-Rigid Connections

As mentioned above, neither BS5950:Pt 3 nor EC4 have made recommendations to enable account to be taken of semi-rigid action in design of composite frames. The hints made in EC4 are not enough to permit designers to perform adequate calculations nor analyse rigorously a composite frame (or member) with semi-rigid connections. There is need for both experimental and analytical study on the behaviour of joints in composite structures. Further research should aim at the methods for prediction of connection resistance and stiffness, as well as its effects on composite frame behaviour.

At present, the design of composite building structures is carried out by the simple methods. The beams are usually assumed simply supported and designed according to the required resistance in midspan. Sometimes though, mainly in bridge design, continuous beams are designed. In this case, sufficient reinforcement is provided to assume that the beams act continuously over the internal supports.

To verify continuous composite beams for the ultimate limit states, the members may be analysed by elastic, or subject to certain conditions, by plastic methods. The appropriateness of a method depends on the ductility of the reinforcement and on the susceptibility of the steel section to local buckling.

In early stages of loading, the beam behaves elastically, with the moments at the internal supports different to those in the midspan region. If the adjacent spans are loaded, the moment at the internal support will be greater than that at midspan. Since the plastic moment resistance in negative bending is normally smaller than that in positive bending, a large amount of rotation capacity is required at internal supports to achieve a complete plastic hinge mechanism. The beams in a composite structure are supported at the ends by composite connections and continuity is provided by longitudinal reinforcement. Therefore two important characteristics in the design of composite connections should be considered: the moment resistance and the rotation capacity. There is no established rule for prediction of these aspects of composite joints and a need exists for research on these issues. The moment resistance, stiffness and the rotation capacity of composite joints are the subjects of Part Two of this thesis.

Even if sufficient rotation capacity is not available to achieve a plastic collapse mechanism, some redistribution of moment will usually occur from internal supports due to cracking of concrete and yielding of reinforcement and the steel section. Elastic analysis can be based on either the gross section or cracked section. The permitted redistribution is dependent on the method of global analysis used to calculate the internal moments. The redistribution is also dependent on the susceptibility of the steel section to local buckling.

Both BS5950:Pt 3.1 and EC4 have recommended maximum percentages of support moment that can be redistributed but these are only applicable to fully-continuous structures. There is need for more research on this matter with regard to the rotation capacity of semi-rigid connections.

1-5-Scope of Part Two

The need for research on the behaviour of composite connections has been discussed above. In comparison with steel connections, much fewer studies have been made on composite joints. The codes of practice for composite construction which have allowed semi-continuous framing lack explicit rules to take account of semi-rigid composite connections. The experimental project described in Part Two concerned the behaviour of composite connections with steel end plate joints. This work is followed by studies on the moment redistribution and required rotation of composite beams with semi-rigid connections.

In order to specify a test programme, a review was necessary concerning the available data. Chapter 4 describes the previous tests on composite beam-to-column connections. The review of Zandonini(1989) covers 26 experiments, but the author's survey includes 88 experiments. The comprehensive background given in Chapter 4 can be a useful reference for further research.

The testing procedures and observations of eleven full scale tests are described in Chapter 5, while Chapter 6 is devoted to the assessment of the experimental results. The measurement methods are described fully and assessed in the light of experience. The testing procedures are explained in detail and the results obtained are analysed individually and also in a comparative manner. Design features resulting from the tests are highlighted for practical applications.

The results of the experimental work have been used to propose a prediction method for the stiffness of composite connections with steel end plate joints. Thus, a brief review on the work done by other researchers on this subject is given in Chapter 7. A mechanical model is shown to be preferable for composite joints. Most of the previous prediction models assume that the plane sections remain plane under applied moments, i.e. they do not take account of the connectors' flexibility. It is demonstrated in Chapter 7 that the neglect of this effect results in an overestimation of connection stiffness and therefore it has to be included in any model.

The research reported in Chapters 4 to 7 provides material of including connection effects in the analysis of composite members. In order to incorporate these effects, a computer program has been developed to analyse a beam with semi-rigid joints at its ends, to represent a beam member of a semi-rigid composite frame. This program is described in Chapter 8 and has been used to analyse composite beams with variable parameters. The results of this numerical study are presented and compared with the test results. Formulae are proposed for calculation of the required rotation of the joints in order to develop a plastic mechanism in the beam. A sample chart is also given to be alternatively used for determination of the required rotation. The results of Chapter 8 form a basis for the choice of the connection type and components, as well as the extent of moment redistribution from the joints to the midspan.

Eventually, Chapter 9 will summarize the concluding remarks of the thesis and gives suggestions for further study on the above subjects.

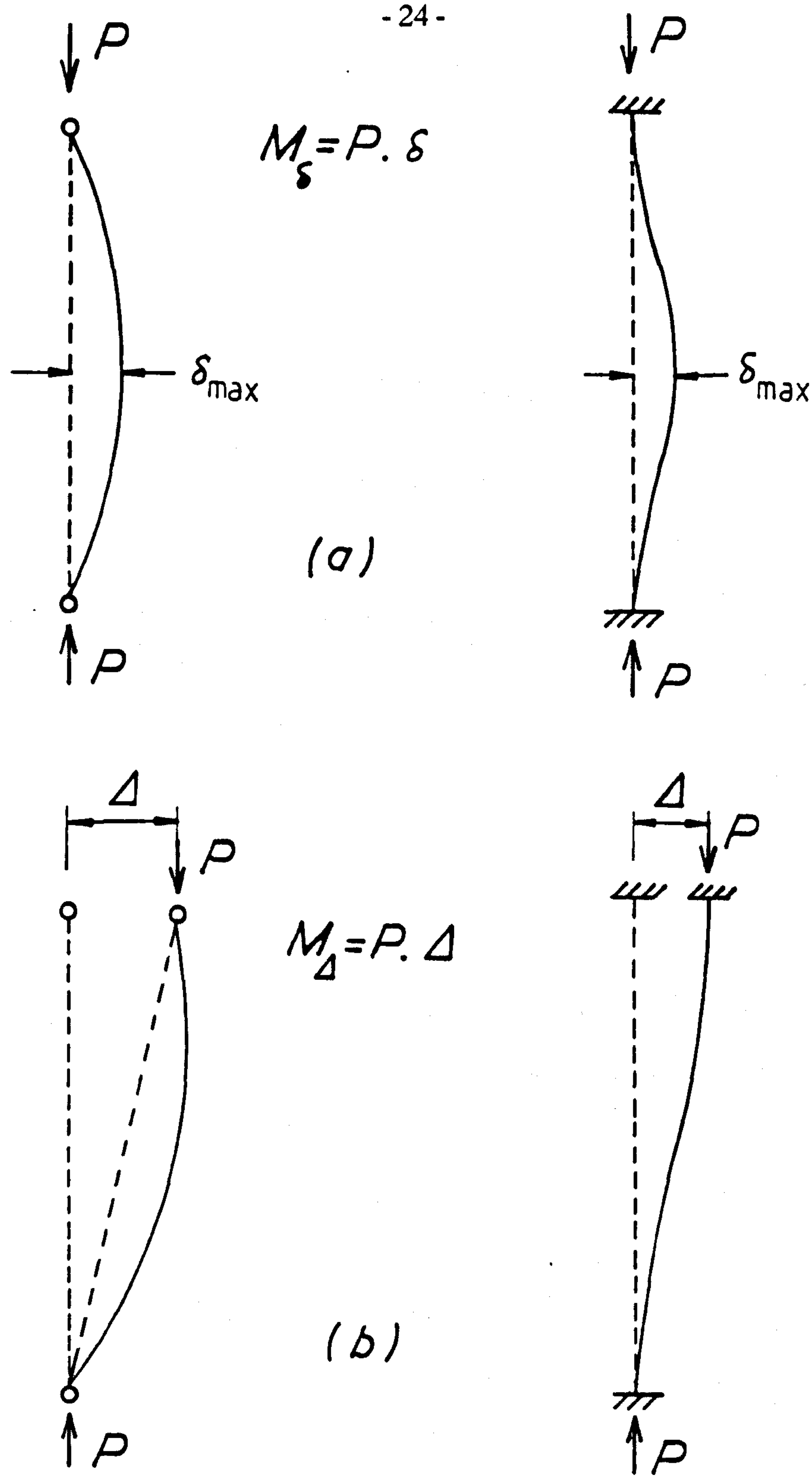


Fig. 1-1, The P–δ and P–Δ effects

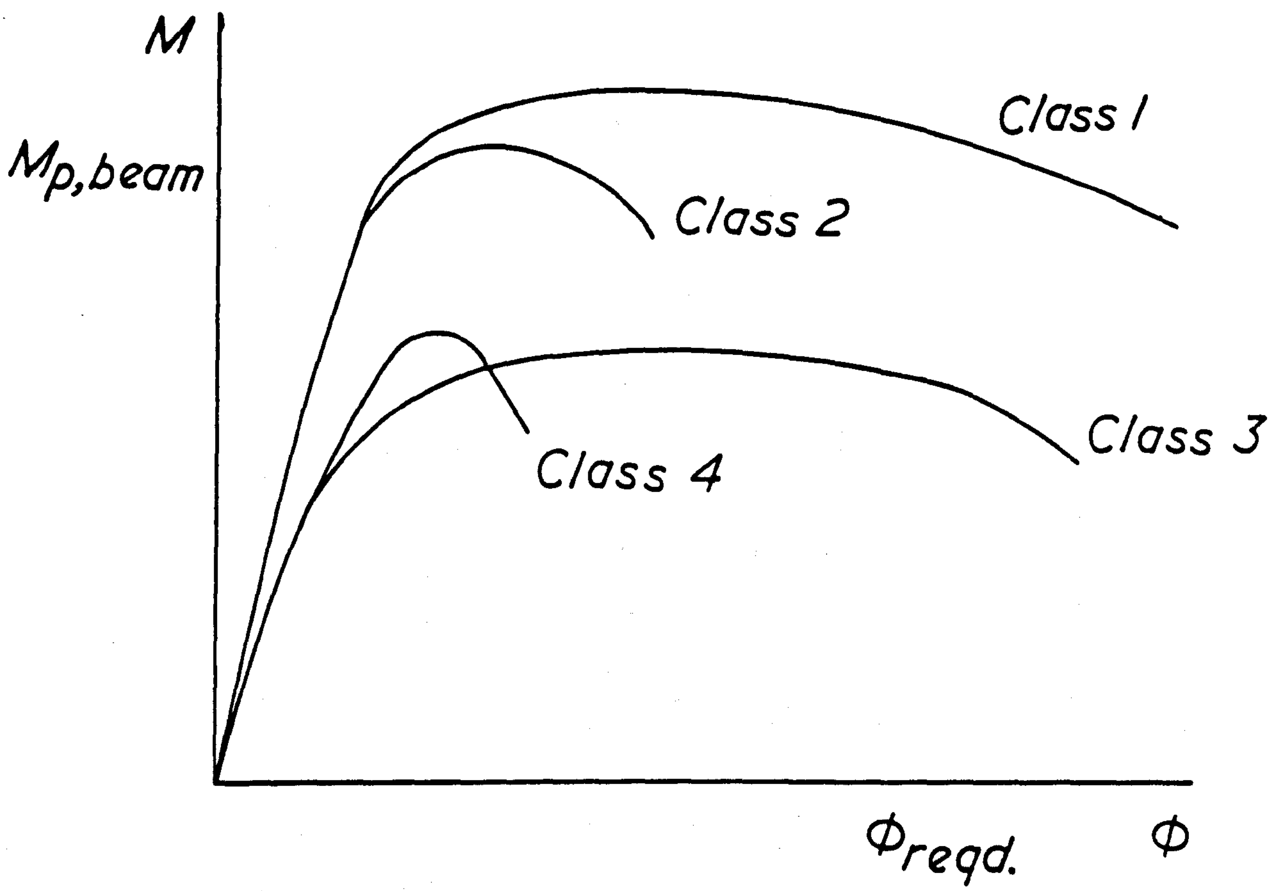


Fig. 1-2, Classification of steel sections

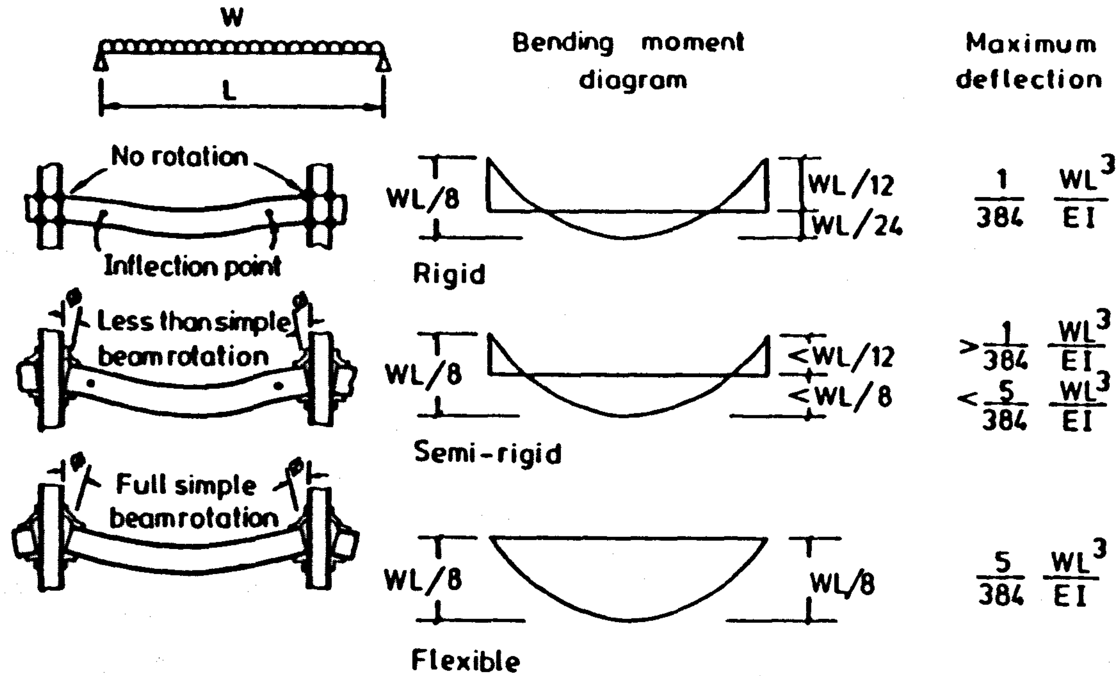
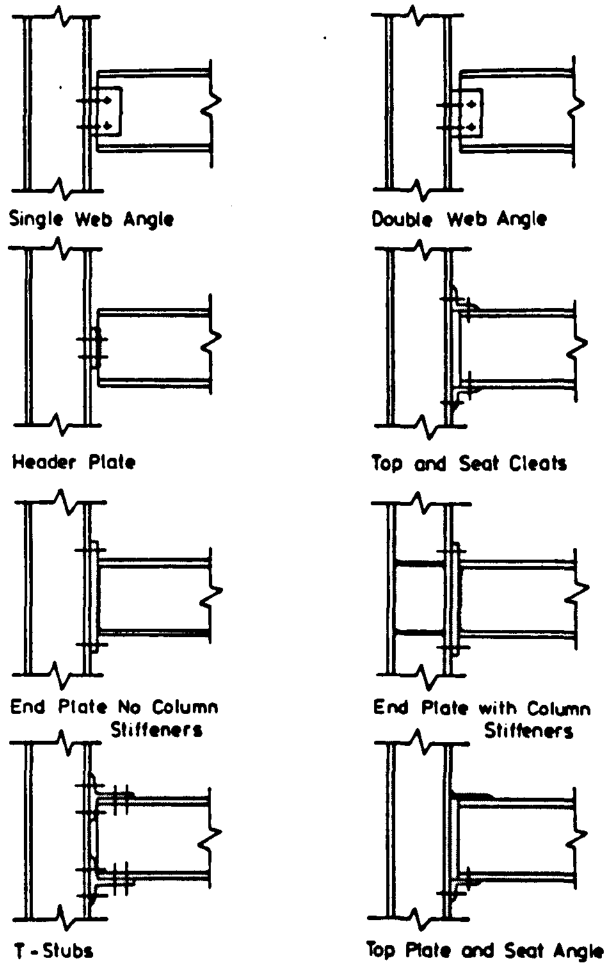


Fig. 1-3, Effect of end restraint on moments and deflections of a beam framed to columns



Common connection types

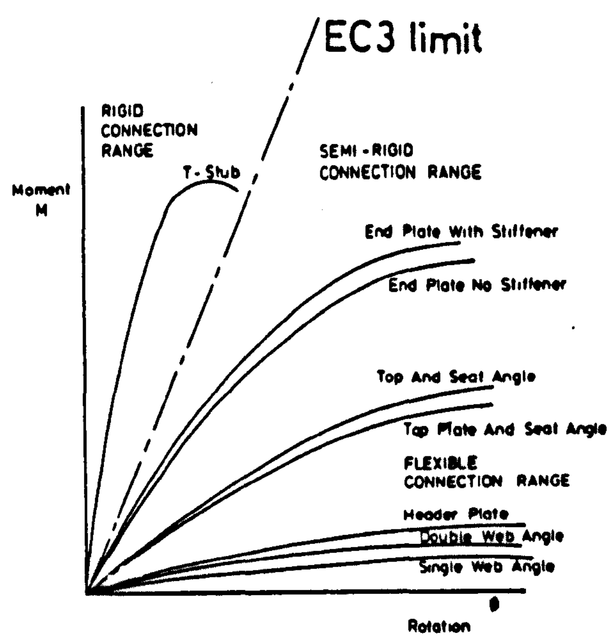
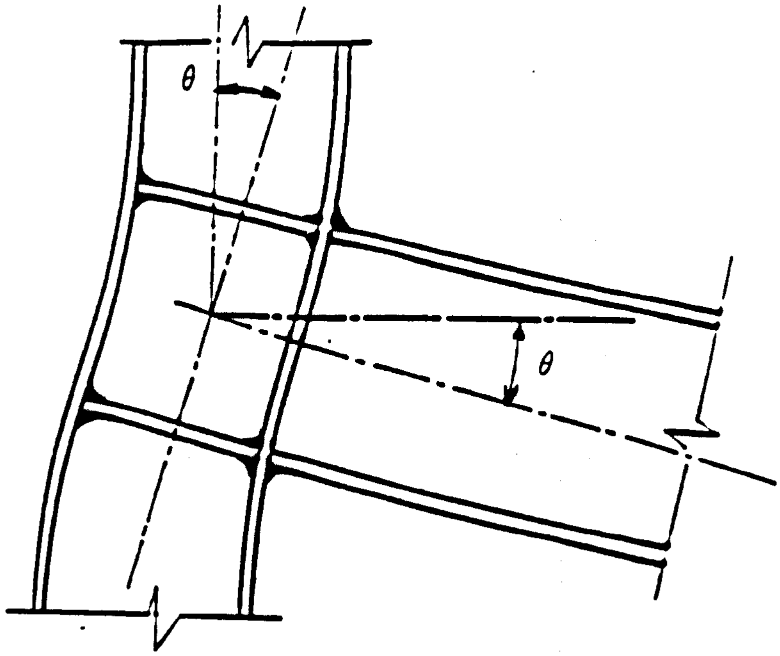
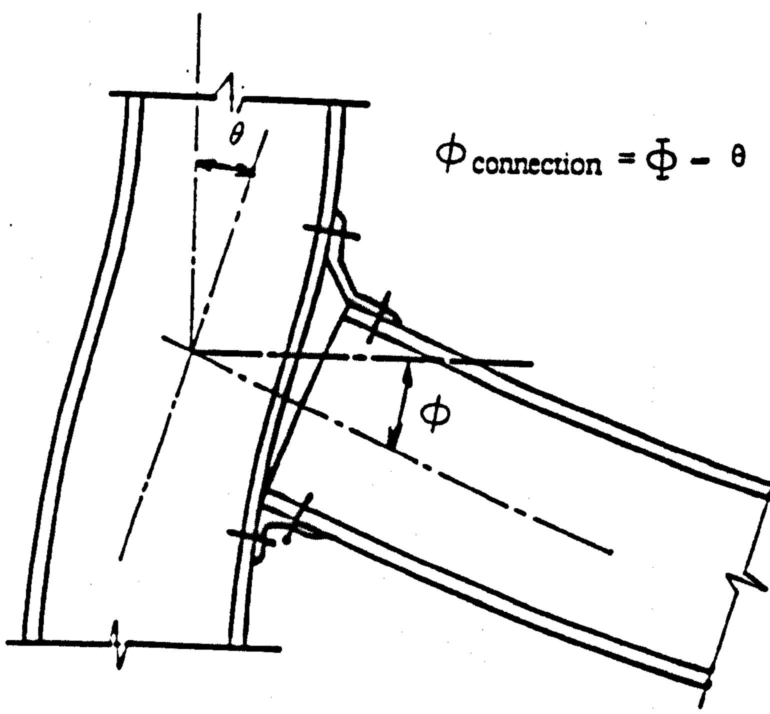


Fig. 1-4, Typical connection types and moment-rotation curves (Jones et al(1980))



(a) Rigid



(b) Semi-rigid

Fig. 1-5, Definition of connection rotation, (a)rigid and (b)semi-rigid

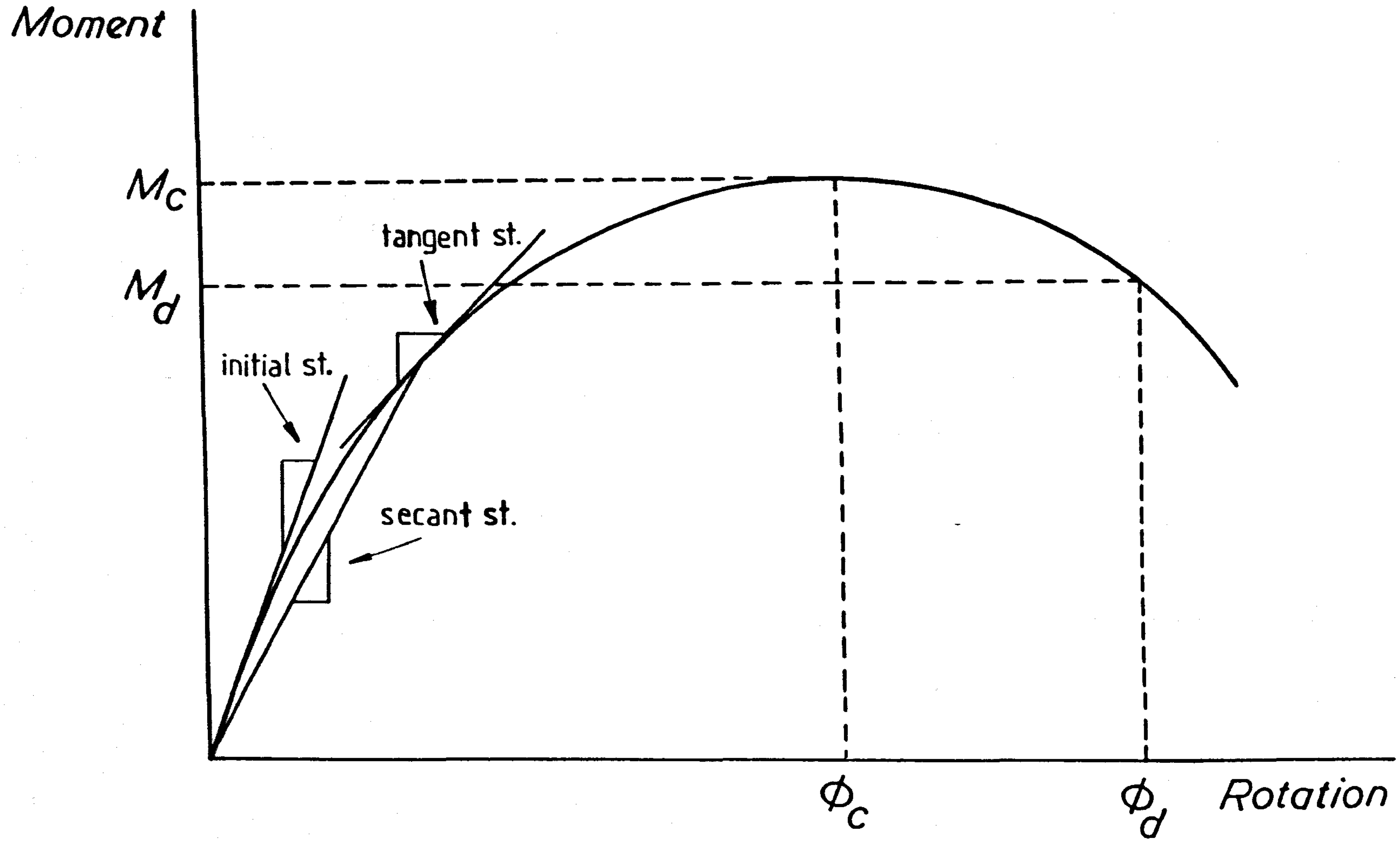
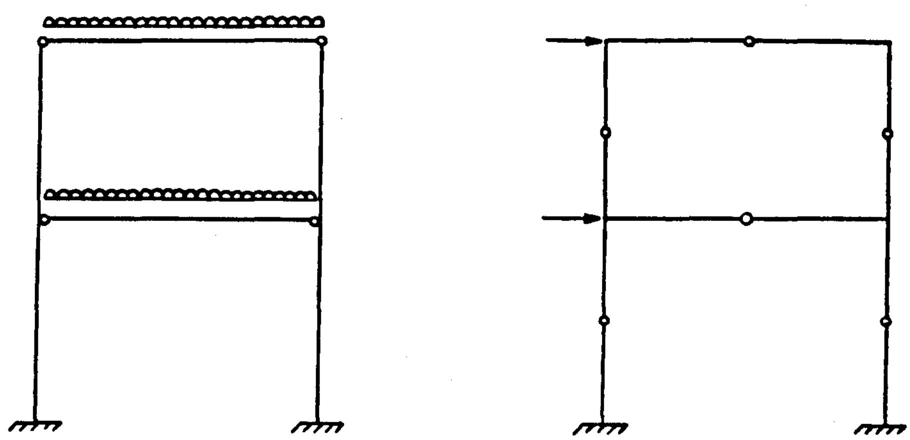
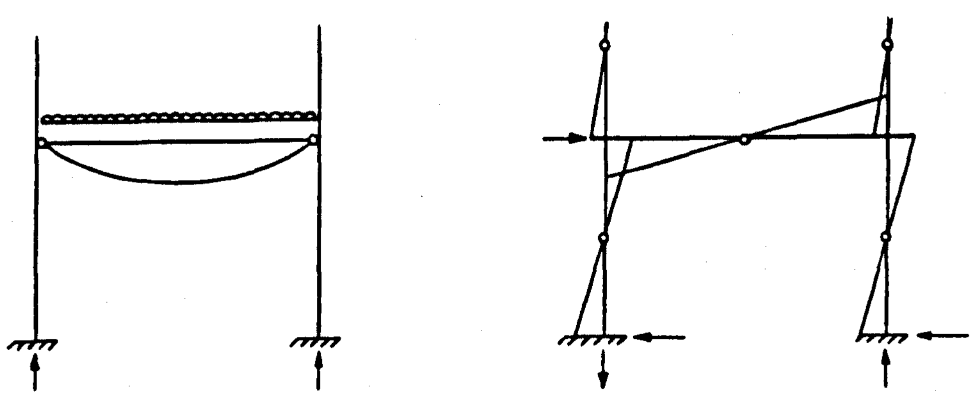


Fig. 1-6, Definitions of moment resistance M_c, design moment M_d, rotation capacity ϕ_d and rotational stiffnesses



(a) FRAME UNDER GRAVITY LOAD (b) FRAME UNDER WIND LOAD

Fig. 1-7, Frame idealization for wind-moment method



(a) GRAVITY LOAD (b) WIND LOAD

Fig. 1-8, Internal moments and forces according to wind-moment method

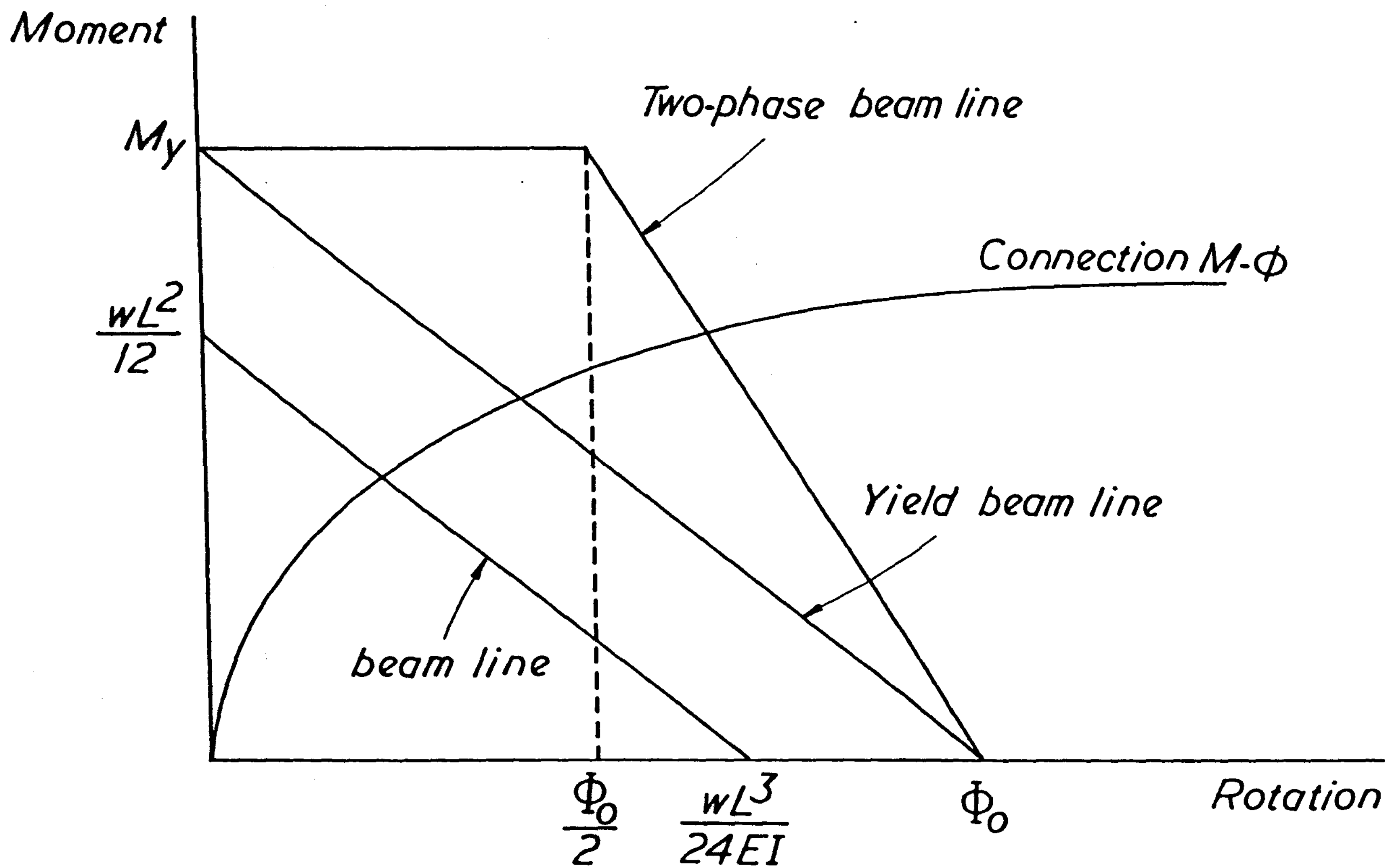


Fig. 1-9, The beam line method

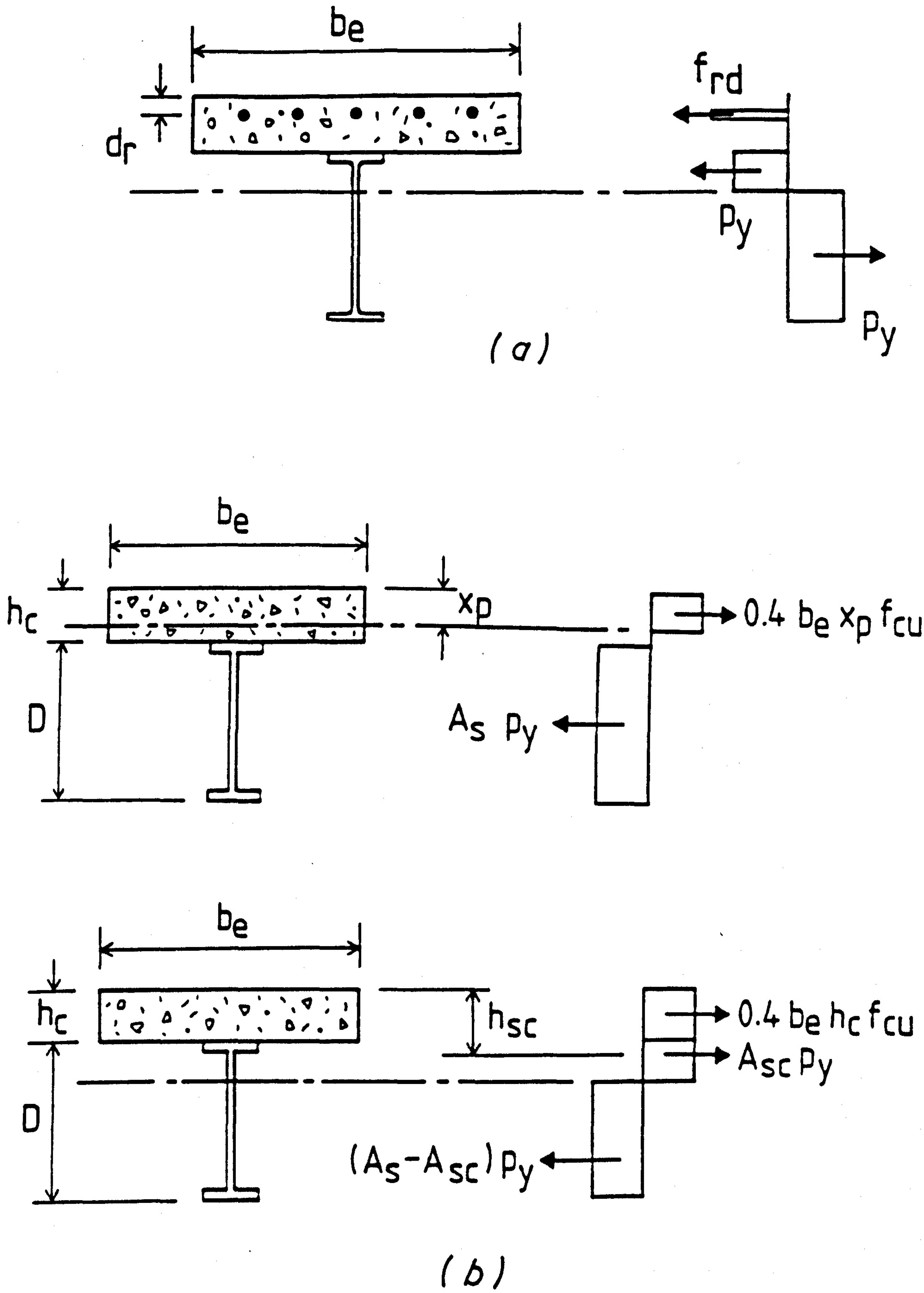


Fig. 1-10, Stress distribution in composite sections, (a)negative moment, (b)positive moment with plastic neutral axis in concrete flange and steel beam

PART ONE

END PLATE CONNECTIONS
IN UNBRACED STEEL FRAMES

Chapter 2
INTRODUCTION TO SEMI-RIGID STEEL CONNECTIONS

The efforts made by researchers to clarify the significance of the rotational behaviour of joints in semi-rigid frames are summarized herein. Extensive experimental and analytical studies have been undertaken over several decades. The results of the tests on steel joints have been used by investigators to derive prediction equations for the connection stiffness. The effects of semi-rigid joints on the frame behaviour in both serviceability and ultimate states have been studied by these researchers and relevant suggestions have been made.

This brief literature survey consists of four parts. Firstly, a review will be given of the experimental studies on steel connections, with particular attention to end plate connections. This is accompanied by a summary of research to predict the behaviour of such connections. Secondly, a more general description is given of the different approaches used to model the moment-rotation characteristics of semi-rigid connections. Thirdly, the investigations on the response of steel frames with semi-rigid connections are explained. Finally, the recent studies on the effects of semi-rigid connections on the stiffness of columns are summarized.

2-1-Experimental Background to Semi-Rigid Connections

The importance of the degree of end restraint provided by semi-rigid connections was realized over seventy years ago. Since then, several hundred connections have been tested to find their actual behaviour.

Three major reviews were made on experimental results of semi-rigid joints by Goverdhan(1984), Nethercot(1985a), and Kishi & Chen(1986). Details of connection behaviour have also been gathered by Bijlaard et al(1987). Benterkia(1991) performed a data collection on the behaviour of end plate connections. Most of the available data of $M-\phi$ relationships are for connections to the flange of the column rather than to the web. Research centres have provided updated experimental information of semi-rigid joints in the form of data bases.

Wilson & Moore(1917) first investigated the flexibility of riveted structural connections. Between 1929 and 1936, the Steel Structures Research Committee (1931)(1934)(1936) investigated several aspects of steel connections to establish a basis for analysis of steel structures. The work undertaken by the Committee formed the basis of the BS449(1969) provisions to incorporate the semi-rigid end restraint into the analysis of steel frames. From these investigations it was realized that economy was possible by reducing the moment and deflection at the centre of a beam.

An enormous number of tests have been carried out on steel joints since then. The end plate connections are those which are the subject of this thesis. Thus, the studies on these types of connection are of most interest in this chapter.

Three types of end plate connections are commonly used in practice. These are the following (shown in Fig. 2-1):

1) Header plate connection.

2) Flush end plate connection.

3) Extended end plate connection.

The header plate connection acts principally as a shear type connection to transfer only vertical shear, whilst the second and third, especially where used with column stiffeners, possess considerable moment resistance. Most of the following statements on the end plate connections have been made by Benterkia(1991) from which further details can be obtained. He studied end plate steel connections bolted to the flange of the column. In this section, it is these types of joint that are referred to, unless otherwise stated.

2-1-1-Header Plate Connection

A summary of the tests conducted on header plate connections is given in Table 2-1. The end plate depth in these tests varies between 40% and 80% of the beam depth. The moment transmitted by these connections ranges from 5% to 25% of the yield moment of the beam.

The behaviour of this type of connection consists of two distinct phases (Fig. 2-2). The transition between these phases occur when the bottom flange of the beam rotates sufficiently that it bears directly against the column face. Thus, their $M-\phi$ curves exhibit a sudden increase in stiffness following the initial non-linear response.

Three models have been introduced by Sommer(1969), Ang & Morris(1984) and Kriviak & Kennedy(1984). The last model has been found to have considerable error in the working range of connections, while the other two provide a realistic representation of connection behaviour up to rotations about 25 mrad for end plate thickness less than 12mm.

2-1-2-Flush End Plate Connection

The reported tests on flush end plate connections are summarized in Table 2-2. These connections transfer much greater amounts of moment, compared to header plates, depending particularly on the thickness of the connected plys of the joint. This moment is usually less than the resistance moment of the beam, unless a thick end plate and column web stiffeners are provided. Thus, they are generally considered as semi-rigid partial strength joints. Their moment-rotation behaviour is continuously non-linear (Fig. 2-2).

Frye & Morris(1975), Johnson & Law(1981), Kukreti et al(1987) and recently Benterkia(1991) have proposed prediction equations for end plate connections. Frye and Morris extended the work of Sommer(1969). They have given the same equations for flush and extended end plate connections but they distinguish between stiffened and unstiffened columns. Their equations have been found inaccurate, hence suggestions have been made to improve them. These will be discussed later in Chapter 6.

The prediction equation of Johnson and Law with stiffened column is based on the evaluation of the initial stiffness and the ultimate moment capacity of the connection. It consists of two straight lines which represent a rough approach to the $M-\phi$ curve. The prediction of the initial connection stiffness is unsatisfactory and inadequate for end plate thinner than or of the same thickness as the column flange. For failure modes other than yielding of the column flange, the predicted moment capacity is in considerable error.

The model suggested by Kukreti et al is based on finite elements. From comparison with 8 tests, they reported a difference ranging from -5% to 20% between the predicted and the test values. The negative sign indicates lesser predicted value.

The equation proposed by Benterkia is for an unstiffened column and has been claimed to be reasonably accurate for connections with end plate thickness greater than 6mm and a beam depth between 250 and 400mm.

In addition to the major axis tests tabulated in Table 2-2, a series of 22 tests have been carried out by Kim(1988) on flush end plate connections bolted to the column web. The main variables in his tests were the end plate thickness, the bolt numbers, the sizes of the beam and column. Kim has plotted the experimental $M-\phi$ data, but his model which is based on the yield line pattern in the column web cannot be used without fundamental changes.

Celikag(1990) has also reported tests on major and minor axis flush end plate connections which were subsequently used in the column and frame tests reported by Gibbons(1990).

2-1-3-Extended End Plate Connection

The available tests on extended end plate connections are tabulated in Table 2-3. It is seen that this type of connection has received the widest study. Most of the data is for end plates extended on the tension side only. Different degrees of moment transfer have been reported. The extended end plate connections possess generally more resistance and stiffness compared to similar flush end plate connections. However, they do not exhibit the rigid behaviour assumed in a conventional analysis. Their moment-rotation behaviour is non-linear over almost entire range of the loading. The initial part of their $M-\phi$ curve may be assumed linear up to about 50% of their moment resistance (Fig. 2-2).

Frye & Morris(1975), Krishnamurthy et al(1979), Tarpy & Cardinal(1981) and more recently Yee & Melchers(1986) have suggested prediction models for the behaviour of such connections. Tschemmernegg & Humer(1988) have recently used a mechanical model analysed by finite element method for extended end plate joints. The Frye & Morris model has been discussed earlier and will be considered in more detail later in Chapters 3 and 6.

The model of Krishnamurthy et al is based on a finite element analysis. It has been given for very thick or stiffened column flanges and symmetrical end plate connections. Otherwise, the rotation contributions by the two halves of the connection must be calculated separately and added. The column flange behaviour is not included in their model and therefore is not suitable for unstiffened extended end plate connections.

The Tarpy & Cardinal equation is for unstiffened connections. It has been generated from a finite element analysis based on a linear elastic model. This equation simulates the connection behaviour in a very stiff manner. They provided also an equation for the moment resistance of the joint which has been found to be considerably in error.

The non-linear mathematical model of Yee and Melchers differentiates between stiffened and unstiffened columns. They only considered the configuration of four bolts around the tension flange, for which they reported a good agreement with the test data from 16 specimens. Their equation has been critisized by Maquoi & Jaspart(1987) based on the arguement that the exponential term of the equation is dimensionless and therefore needs to include a term which is in load-length dimension (not the dimensionless parameter as presently exists) to give a value in moment units.

In the model of Tschemmernegg and Humer the "connection means" which are the sources of "connection" behaviour are welds, the end plate, bolts and the column flange. The load-deformation response of these means should be superimposed to that of the "panel zone" which consists of the column web and the two flanges of the column. The result will be the overall behaviour of the "joint". Their investigation concentrated on the tension zone of the joint, the capacity of which can be adjusted to the compressive capacity of the panel zone. They have then developed tables for standardized connections. The author has not found an independent source justifying this approach.

2-2-Methods of Prediction of Joint Behaviour

A major review on the prediction methods of the behaviour of semi-rigid connections has been made by Nethercot & Zandonini(1989). The shape of the moment-rotation curve of connections proposed by researchers are either linear or non-linear, based on the following models:

a) Empirical curve fitting model; which is the gathering of data on moment-rotation curves of connections and employing curve fitting and regression techniques.

b) Analytical model; which is developed in two phases, firstly assuming simplified models for the main parameters of the $M-\phi$ curve, and secondly a curve fitting procedure to develop the full $M-\phi$ relationship.

c) Mechanical model; which considers the deformation behaviour of the connection elements by employing the load-deformation characteristic of materials.

d) Finite element model; in which the connection components are divided into a number of elements with common boundary conditions at their interface and suitable material properties are used to find the elastic, elastic-plastic and the collapse behaviour of the connection.

The expressions derived by investigators give different domains of connection stiffness which can be categorized as following (see Fig. 2-3):

1) Initial stiffness; which is the initial tangent stiffness of the moment-rotation behaviour of the joint.

2) Bilinear; which consists of two straight lines. The first line represents the secant stiffness of a point on the actual $M-\phi$ curve of the connection about which the stiffness varies significantly. The second line is either horizontal to represent a plateau for the $M-\phi$ curve, or has a small slope.

3) Piece-wise linear; which is a mutilinear representation of connection behaviour consisting of several lines assumed to circumscribe the actual $M-\phi$ curve of the joint.

4) Non-linear; which is in the form of a curve which can result from mathematical curve fitting techniques, mechanical models or finite element methods.

The work undertaken by research workers which resulted in the above types of representation will be summarized in this section. Furthermore, the investigations using mechanical and finite element models will be described. Finally, the moment-rotation characteristic adopted by EC3 will be given.

2-2-1-Initial Stiffness

The initial stiffness was mainly used in early studies on semi-rigid connections to derive a mathematical representation of the moment-rotation behaviour. Baker(1934), Rathburn(1936) and later Lothers(1951) expressed the initial stiffness in the linear form of a tangent to the initial slope given by:

$$Z = \frac{\phi}{M} \tag{2.1}$$

where Z is the rotation of the joint for a unit value of moment. This expression may be used directly into the slope-deflection equations or the moment distribution method. An analytical procedure has been developed recently by Azizinamini & Radziminski(1987) to predict the initial stiffness of cleated connections.

Since the initial tangent to the $M-\phi$ curve does not account for the reduced stiffness of the joint at higher moments, it overestimates the connection stiffness generally and can only be acceptable at very low load levels.

2-2-2-Bilinear

Lionberger & Weaver(1969) and Romstad & Subramanian(1970) introduced a bilinear representation which recognized the loss of stiffness at higher rotations. The model of Johnson & Law(1981) was also bilinear. This type is a rough approximation of connection behaviour and is acceptable for certain types of connections (Nethercot & Zandonini(1989)). It underestimates both the connection stiffness and resistance because of being beneath the actual curve.

Zandonini & Zanon(1988) proposed simplified bilinear and trilinear representations of the $M-\phi$ curve. Such prediction methods, capable of handling only the key aspects of connection behaviour, are useful because they simplify design procedures.

2-2-3-Piece-wise Linear

Better approximations of the $M-\phi$ curve are obtained by increasing the number of linear portions in the multilinear representation of joint behaviour. A trilinear model has been presented by Moncarz & Gerstle(1981), and a quadrilinear model by Melchers & Kaur(1982). Poggi & Zandonini(1985) have suggested a multilinear approximation of connection behaviour to be used in numerical analyses, but they give no formula for calculation of the piece-wise representation.

This type of moment-rotation curve can be usefully employed in computer programs for analysis of semi-rigid frames. It can be treated by an analysis performed as a sequence of linear steps.

2-2-4-Non-Linear

Polynomial representations were suggested by Kennedy(1969) and Sommer(1969), thereby introducing the curved nature of the $M-\phi$ relationship.

Non-linear $M-\phi$ relationships were obtained by curve fitting to experimental data. Frye & Morris(1975) used a standardized format to express rotation in terms of moment via several constants. The general relationship can be shown as:

$$\phi = \sum_{i=0}^{i=m} C_i (KM)^i \tag{2.2}$$

where C_i are constants and K is the standardization factor. This relationship will be discussed further in Chapter 3.

Jones et al(1982) divided the experimental data into a number of subsets, each spanning a small range of moment. A Cubic B-Spline curve was then used to fit each and every subset of data.

Ang & Morris(1984) used the Ramberg-Osgood(1943) function to model the non-linear behaviour of connections, which modified the earlier proposals of Frye & Morris(1975).

Yee & Melchers(1986) proposed an exponential model. Krishnamurthy et al(1979) and Kukreti et al(1987) gave power models. Benterkia(1991) evaluated the existing prediction equations for end plate joints and proposed a power model for unstiffened flush end plate connections based on curve fitting of the available data.

2-2-5-Mechanical Models

Wales & Rossow(1983) and Kennedy & Hafez(1984) developed mechanical models for header plate and web cleat connections respectively as shown in Figs. 2-4 and 2-5. The spring model was later extended by Chmielowiec & Richard(1987) to predict the behaviour of all types of cleated connections subjected to bending and shear (Fig. 2-6).

Tschemmernegg(1988) represents welded and bolted joints by mechanical models as shown in Fig. 2-7. Springs A are meant to account for the load introduction effect from the beam to the column, while springs B simulate the shear flexibility of the column web panel. Springs C allow for the additional sources of deformation present in bolted connections. The spring characteristics are defined and the overall behaviour of the steel joint can be determined by superposition of the response of each group of springs.

In general, mechanical models have been confirmed as an adequate and promising tool for the study of steel connections (Nethercot & Zandonini(1989)). These models need the mechanical behaviour of materials and therefore their accuracy relies on the accuracy of the assumed material properties. The understanding of the individual and interactive behaviour of the connection components is necessary. Such behaviour is dependent on geometrical and mechanical parameters of the joints.

2-2-6-Finite Element Models

The first use of a finite element model was related to welded beam-to-column connections (Bose et al(1972)), in which the column web was given particular attention as the critical component. The agreement with available experimental results encouraged researchers to use this method further. For instance Patel & Chen(1984) and Atamiaz Sibai & Frey(1988) used this method for welded beam-to-column joints.

The non-linear equations of Krishnamurthy et al(1979), Tarpy & Cardinal(1981) and Kukreti et al(1987) mentioned in 2-1 were derived from finite element analysis of end plate connections.

Generally, finite element methods seem to be suitable for predicting the response of welded connections, but a direct analysis of bolted connections requires the ability to model the bolt action (Nethercot & Zandonini(1989)).

2-2-7-Eurocode 3

EC3 recommends that the determination of the design moment-rotation characteristics of beam-to-column connections be based on a theory which is supported by experimental evidence. Although the actual $M-\phi$ curve of connection is recognized by the code as non-linear, appropriate linearized approximations are also acceptable to EC3. The restriction is that the approximate behaviour lies wholly below the more precise characteristic (see Fig. 2-8).

The determination of three main properties of connections are necessary. These are the moment resistance, the rotational stiffness and the rotation capacity. Bilinear and trilinear representations of connection behaviour are acceptable to EC3 with these defined properties.

Annex J of EC3 gives a formula for the stiffness of end plate connections. The moment-rotation curve of the connection is proposed to be taken as linear up to two-thirds of the joint design moment resistance, while its curved shape up to the design moment resistance of the joint can also be approximated by a second straight line. The method assumes a plateau for the curve, i.e. increasing rotation at a constant moment, when the design moment resistance of the connection is reached. This method will be discussed further in Chapter 3.

2-3-Semi-Rigid Analysis of Frames

The analysis of frames with semi-rigid connections have been reviewed by Jones et al(1983) in a "state of the art" report. Anderson et al(1987) have later provided a reference to the design and analysis of steel frames with semi-rigid joints. A comprehensive report has been published recently by ECCS(1992) which is concerned principally with the application of the existing knowledge of joint behaviour to the analysis and design of steel frames.

The effect of semi-rigid connections on the structural members was taken into account by Baker(1936) and Rathburn(1936). They used Eqn. (2.1) to modify the slope-deflection equations. They also modified the stiffness, carry-over factors and the fixed-end moments in the moment distribution method. They assumed a linear moment-rotation relationship which is only applicable for very low values of rotation and becomes increasingly inaccurate as the moment increases, unless corrected to a secant stiffness.

The application of computers has permitted more refined and accurate representations of the joint to be included in analysis. The main methods based on the treatment of joint response (described in 2-2) are:

a) Linear elastic analysis.

b) Non-linear elastic analysis.

c) Inelastic analysis.

Elastic models ignore the possible occurance of joint unloading, while inelastic models include this effect. The second-order effects and material non-linearities may be accounted for in any of these analyses, but previous research mainly included these effects in (b) or (c). The stiffness value used is one of the following:

1) the initial stiffness,

2) the tangent stiffness at any point,

3) the secant stiffness.

The methods of analysis will be explained in the following sub-sections together with reference to relevant investigations carried out by researchers. An indication of the effect of semi-rigid joints on frame behaviour will be given by summarizing the results of analysis of a frame which will later be used in Chapter 3.

2-3-1-Linear Elastic Analysis

This method can be considered as the most consistent with routine design practice. Rigid frame computer programs can be directly used by means of a ficticious beam model in which the stiffness of beam is reduced to allow for the connection flexibility. Alternatively, a spring model may be adopted with modifications in the slope-deflection equations. Due alterations must then be made in the stiffness matrix of a member and the overall stiffness matrix of the frame. The correction coefficients to be applied to the member stiffness are given in Fig. 2-9.

The proper joint stiffness may be selected from one of those described in Chapter 1; i.e. initial, secant or tangent. A number of investigators including Monforton & Wu(1963), Goble(1963), Lightfoot & Le Messurier(1974) incorporated the effects of connection deformations into a stiffness analysis computer program using the initial stiffness of the joints. The initial stiffness though can be assumed only when joint rotations are likely to be very small. Instead, a reduced joint stiffness (effectively a secant stiffness) introduced by Moncarz & Gerstle(1981) may be used which has recently been supported by Jaspart & Maquoi(1989).

2-3-2-Non-Linear Elastic Analysis

The non-linear assumption for moment-rotation behaviour becomes necessary where a closer approximation is required for design to limit states. The non-linear joint behaviour can be allowed for without a substantial additional burden compared to routine rigid frame design.

A piece-wise linear or a curvilinear relation for the connection behaviour may be used with iterative procedures based on the secant stiffness approach. The secant stiffness approach, compared to initial and tangent stiffness approaches, has the advantage of keeping the total error to a minimum at the limit states.

Romstad & Subramanian(1970) analysed frames using variable tangent stiffness of the joints. This method uses the last obtained values of moments to find an appropriate tangent stiffness, and then iterates on the tangent stiffness until an acceptable tolerance is met.

Frye & Morris(1975) presented an iterative analysis procedure for planar steel frames involving repeated cycles of linear analysis incorporating the non-linear connection effects. They used the polynomial equations they had proposed for different types of beam-to-column connections.

Ang & Morris(1984) generalised the Frye & Morris(1975) procedure to analyse three dimensional steel frames. The $P-\Delta$ effect was also included in their analyses.

Moncarz & Gerstle(1981) developed a method of frame analysis which accounted for the non-linear connection behaviour and variable load histories. They examined a number of frames with the different types of construction defined by the AISC Manual(1986). They found that the assumption of rigid joints was inadvisable due to the underestimation of sway deflections.

When joint flexibility is incorporated into a matrix displacement method of analysis, the size of the stiffness matrix increases. In a technique proposed by Anderson & Lok(1983), the deformations of the joints were allowed for by revising the load vector at each iteration. Convergence problems were experienced in their program, so it was later modified by Benterkia(1991) to use successive estimates of the secant stiffness of each connection to ease convergence. Both these programs were based on one for analysis of rigid frames written by Majid & Anderson(1968).

2-3-3-Inelastic Analysis

Procedures for inelastic analysis of semi-rigid frames have been developed mostly in 1980's. An accurate representation of the joint behaviour is generally incorporated into established methods for the elastic-plastic analysis of rigid frames. Because of the non-linear behaviour of connections and the inelastic behaviour of materials, sophisticated numerical approaches are required using iterative procedures. These methods have not yet been commonly employed in design offices although some attempts have been made in this regard including the work by Edinger(1983).

Ackroyd & Gerstle(1983) used the secant stiffnesses of joints, and both material and connection non-linearities in their analysis to find the ultimate strength of the frame under increasing load.

Zandonini(1986) reported an investigation aimed at the stability of flexible frames. A numerical study was carried out on portal frames with different overall stiffnesses. Several modes of failure were recognised. Except for two frames, all collapsed by inelastic sidesway instability and not by the development of a full plastic collapse mechanism. It was concluded that the connection behaviour influences the frame response more significantly when partial strength connections are used compared to full strength connections. In this case, a high rotation capacity may be required. Full strength connection, if the additional cost is justified, reduces the need for connection ductility and provides an adequate frame performance. The accuracy of the $M-\phi$ curve was found to be of influence at SLS but not at ULS.

Ohta(1988) employed one dimensional finite elements to represent the behaviour of semi-rigid connections in the analysis of steel frames.

Zoetemeijer(1989) discussed the limitations of the different types of connection stiffness and described the effect of joint flexibility on second-order effects. He presented the effect of partial strength and semi-rigid connections on the forces and moments of both braced and unbraced frames. He used the beam line method to review simplifying rules for the analysis of braced frames.

Chen(1989) discussed the behaviour and modeling of semi-rigid joints. He refined the computer model he had proposd to examine the non-linear behaviour of frames. The effect of connection flexibility and the panel zone deformation on semi-rigid frames were also studied.

Scholz(1990) applied an approximate elastic-plastic method of analysis to regular steel sway frames with semi-rigid connections. He defined a factor to enable the designer to find the slenderness ratios of the limiting frame which should be evaluated for each member and connection of the framework. The largest ratio is then carried into an equation, the result of which is used in a multicurve diagram of "frame curves". From this diagram the failure load can be found. This approach is extended to semi-rigid connections by using Eqn. (2.1).

Poggi(1990) developed an elastic-plastic finite element beam model which incorporates joint flexibility. Elements consist of three parts: a central elastic-plastic beam, two rigid bars at ends and a set of non-linear springs of null length between each rigid bar and the beam. Joint behaviour was included by the action of these springs, one for each potential deformation; namely axial, shear and rotational. Linear presentation of force-deformation relationships were used in their program. This program was used at Sheffield University and good agreement was reported between analysis and experimental results (Davison et al(1988)).

Chikho & Kirby(1989) examined the effect of joint flexibility on the lateral deflection of frames using Poggi's program. They used three types of connections ranging from rigid to semi-rigid. In rigid frames, the first plastic hinges formed at the columns' ends and collapse occurred when plastic hinges formed at the midspan of beams. In semi-rigid frames, the plastic hinges formed first at the midspan of the beams and collapse occurred when plastic hinges formed at the columns' ends. The effect of semi-rigid connections was found to be dependent on the following factors: the joint $M-\phi$

curve, the stiffness of the frame's members, the applied horizontal and vertical loads, and whether the frame was sway or non-sway.

Jaspart & Maquoi(1989) proposed a simple design method for sway frames with semi-rigid connections. They examined the ultimate load of frames with reference to Merchant-Rankine formula modified by Wood(1974). The $P-\Delta$ effects were taken into account by means of this formula. They found that design of an unbraced frame is usually governed by check of sway at SLS and a braced frame by the checks on the resistance of individual components at ULS. They proposed replacing the actual $M-\phi$ curve of the joints by a ficticious stiffness less than the initial stiffness, to be used in the slope-deflection equations. The method to find this stiffness is still under investigation by the above researchers. They concluded that the Merchant-Rankine formula is accurate as long as the collapse mechanism is of the combined type, i.e. beam plus column panel mechanism; it is slightly conservative when a beam mechanism is governing; and is largely unsafe when a panel mechanism governs.

Jaspart & Maquoi(1990) extended their study on the application of elastic and plastic analysis to braced semi-rigid frames. The plastic design was considered on a frame with stocky columns to allow for plastic hinge to form at the beam midspan. Good confirmation was achieved in comparison between the collapse load multipliers resulting from hand calculations and a numerical simulation.

Kavianpour(1990) developed a program originally written by Majid & Anderson(1968) for elasto-plastic analysis of rigid frames. He made modifications in the stiffness matrix of the frame to include the rotational behaviour of the connections, based on their secant stiffness. He further developed this program to investigate the response of the structure to cyclic loading. The frames with semi-rigid connections were found to shakedown to their elastic state in the same manner as plastic hinges in rigidly connected frames.

Deierlein(1991) describes a method for modelling semi-rigid connections as an "average" curve of $M-\phi$ relationship. Zero-length rotational springs have been used to simulate the behaviour of major and minor axis connections. The existing $M-\phi$ curves have been calibrated by a normalization procedure and implemented in an inelastic program. This "average" curve was found to result in a sufficiently accurate overall response for the frame.

Zandonini & Zanon(1991) analysed a steel beam with semi-rigid connections at its ends under both serviceabilty load w_s and ultimate load w_u. Its behaviour could be defined by the support moment M, the joint rotation ϕ and the midspan deflection δ. Domains for M, ϕ, δ and w have been worked out and drawn on the same coordinates. They defined the lower design boundary for the $M-\phi$ curve of the connection for both elastic and plastic analysis. A design system then was proposed for semi-rigid joints in non-sway frames.

2-3-4-Effect of Joint Behaviour on Response of Three Storey Frame

In order to study the influence of joint flexibility quantatively, the frame shown in Fig. 2-10(a) has been used by various researchers (including the author). It was analysed at various levels of accuracy, assuming that the joints are extended end plates with backing plates as shown in Fig. 2-10(c), possessing the $M-\phi$ behaviour as in Fig. 2-10(d). This type of joint would be traditionally classified as rigid.

The frame was analysed semi-rigidly with both linear elastic and non-linear elastic joint behaviour. The linear behaviour was based on the initial stiffness. The frame was also analysed rigidly. The drift response of frame is shown in Fig. 2-10(b) where the horizontal deflection is plotted against the load multiplier. The limit of H/300 assumed as reference value for serviceability is also shown on the plot.

The following can be observed (ECCS(1992)):

1) The joint deformation influences the frame stiffness more than the ultimate load resistance. As a consequence, the semi-rigid frame still possesses a sufficient ultimate strength, but it does not meet the limit of H/300.

2) The second-order geometrical effects are noticeably greater for the semi-continuous frame, even if joints are treated as linear elastic.

3) The contribution of joint non-linearity to the frame drift becomes substantial for loads higher than the nominal loads ($\alpha > 1.0$).

4) In the semi-rigid frame, the deterioration of the frame stiffness due to member yielding is immediate and important, leading to a rapid attainment of collapse. The influence of plasticity on the rigid frame's response is more gradual.

The following conclusions can be drawn from this example:

a) Joint flexibility must be evaluated in analysis of frames.

b) A second-order analysis, which assumes elastic member behaviour but accounts for the non-linearity of joint behaviour, allows a sufficiently accurate prediction of both frame stiffness and strength (ECCS(1992)).

c) The assumption of linear joint behaviour may be suitable for checking the frame under working loads.

2-4-Recent Studies on Columns Restrained by Semi-Rigid Connections

For frames classified as non-sway, second-order effects due to sway may be neglected and first-order analysis performed. For sway frames, the effects of horizontal displacements must be accounted for. In order to design columns in sway frames, two approaches are identified. The first approach is to account for the beams by either assuming a greater length for the columns, or amplifying the end moments. The second approach is the "exact" analysis of column within the frame.

The column length taken in the first approach is termed the "effective length" of column. The effective length of a member is defined as that length of a pinned member which has the same buckling load as the member under consideration. For a column, it depends on the boundary conditions at the ends of its unbraced length. In a framed structure, the boundary conditions depend on the stiffness of the beams framed into the column through the connections. Therefore the effect of the stiffness of such connections is a significant factor in determination of the column stiffness.

Idealized solutions for critical loads and effective lengths are shown in Fig. 2-11. The conventional guides for determining the effective length of the columns concern the extreme cases where beams are rigidly connected or pinned to the columns. The same conditions are assumed at both ends. A more accurate design will result if the effects of the stiffness of both beams and connections are taken into account.

The value taken as the effective length of the columns in the design of steel frames can significantly influence the cost of the structure. Bjorhovde(1984) reports that the effect of 10% reduction in the effective length factor k, obtained by stiffer connections, provides a saving in material up to 11%.

2-4-1-Indirect Design of Column

When elastic global analysis is used for a sway frame, second-order effects may be included indirectly by first-order analysis of the frame compled with sway mode effective buckling lengths for the columns. The use of an effective length greater than the true length is an established method to allow for second-order sway effects in rigid jointed frames (see Fig. 2-11(e)). For a semi-rigid frame, the reduction in the effective stiffness of a beam due to flexibility of the joints are allowed for by a correction to the beam stiffness, which can be derived from the slope-deflection formulae.

2-4-2-Background to Columns in Semi-Rigid Frames

A review on the column stability has been made by Anderson et al(1987) in which the existing studies to that date have been described.

De Falco & Marino(1966) developed a chart for determination of the relative stiffness of beams with semi-rigid connections to be used in determining the effective length of columns. Driscoll(1976) revised the chart and presented a general solution for the stiffness of beams with semi-rigid connections. He suggested a simplified solution using Eqn. (2.1).

Sophisticated charts were developed by Wood(1974) which could deal with any local degree of end restraint both for sway and no-sway conditions. Modifications were also made for cladding stiffness.

Jones et al(1980) introduced a computer program for calculation of the strength of columns with semi-rigid end restraint in which initial lack of straightness, spread of yield through the cross section and residual stresses were taken into account. The results of the analyses demonstrated the increase in column strength due to end restraint.

Galambos(1982) suggested that the effective length of a column could be estimated from the stiffness of the minor axis beam-to-column connection.

Bjorhovde(1984) conducted studies on the effect of end restraint on the column strength in sway-prevented frames. He used the initial stiffness of the connection and the beam stiffness to determine the stiffness of interior and exterior columns. Design recommendations have finally been made.

Lui & Chen(1986) proposed relationships for the effective length ratio of the columns in terms of the connection stiffness.

Nethercot et al(1987) and Rifai(1987) studied the behaviour of columns in semi-rigid frames. They developed a non-linear finite element method in which the effect of semi-rigid joints was incorporated to the stiffness matrices of elements. They found that even quite flexible joints raise the ultimate load for the column significantly above that of an equivalent pinned ended one.

Davison(1987) conducted experimental and analytical studies on the semi-rigid steel connections and the behaviour of columns in steel frames. He tested subassemblage and full scale frame specimens. Compared to simple framing, enhanced column resistances were found due to the restraint provided by the connections. During failure of the column, connections on either side rotated in the same direction causing unloading of one end and continued loading of the other. The unloading connection had the greater stiffness and hence restrained the column more than the adjacent, loading connection.

Jones et al(1987) and Nethercot & Chen(1988) have further investigated the effects of the joint flexibility on the stiffness of columns. In the analytical part of their studies, variable end restraint and different column sections and orientation were used. In all these investigations the dominating effect of connection stiffness was emphasised and this effect was considered to be more significant where the slenderness of column increased. In particular, the slenderness factor found from the calculations for major axis non-sway situation of column, was less than those in the codes of practice. This would help the designers to achieve more economy in design.

Nethercot(1991) synthesised the results of recent research on the behaviour of non-sway steel frames with semi-rigid or partial strength joints into a set of design proposals. These cover the behaviour of major and minor axis joints as well as the biaxial behaviour of both internal and external columns. Two basic possibilities of such frames were considered: strong column, weak beam and weak column, strong beam. In the former semi-rigid connections act about the major axis while in the latter about the minor axis. Recommendation is made for a simple approach for problem of biaxial column buckling; namely, take the actual column length in design for major axis buckling; for minor axis buckling the suggestion of Galambos(1982) for column stiffness is to be used:

$$K = \frac{K_{beam} K_{joint}}{K_{beam} + K_{joint}} \tag{2.3}$$

Gibbons(1990) included the effect of column web flexibility into Eqn. (2.3) and proposed a relationship for the total restraint provided to the column.

The EC3(1992) approach to the buckling length of a column is given in Appendix E of the code. The "distribution factors" are obtained from Fig. 2-12(a) and used in the relevant chart for the sway or non-sway mode to find the effective length ratio for the column. The chart for sway mode is given in Fig. 2-12(b).

2-5-Conclusions

In order to achieve economy in design, knowledge of the moment-rotation behaviour of the connections is required. Extensive experimental studies have been undertaken on this subject since the early decades of this century. The studies on end plate connections have been summarized in this chapter.

The difficulty in conducting tests on each configuration of actual joints caused the researchers to derive prediction equations to simulate the behaviour of steel connections. The resulting representations are in various forms: linear, bilinear, multilinear and non-linear. Such predictions have been based on one of the following models:

a) Empirical curve fitting model.

b) Analytical model.

c) Mechanical model.

d) Finite element model.

The effect of semi-rigid connections on the frame response has been studied since 1930's. With the development of computers, comprehensive analyses employing iteration techniques became possible. These include semi-rigid joints, which modify the stiffness matrix. The matrix stiffness method became a popular approach as different aspects of structural properties could be included in the frame analysis.

The serviceability and ultimate limit states of frames with semi-rigid connections have been examined. The M–ϕ curve of connections influences the frame's drift at SLS, but the connections' moment resistance affects the frame response at ULS. The geometric and joint's non-linearities are more influential in the behaviour of semi-rigid frames compared to rigid frames. Hence, joint flexibility must be taken into account in analysis and design of structures.

The restraint provided by semi-rigid joints to the columns has been investigated. The result was that the extreme assumptions of end restraint, namely rigid or pinned, do not reflect the actual column stiffness. A column in a semi-rigid frame possesses an effective length between those corresponding to rigid or pinned ends.

Table 2-1, Available experimental data for header plate connections (after Benterkia(1991))

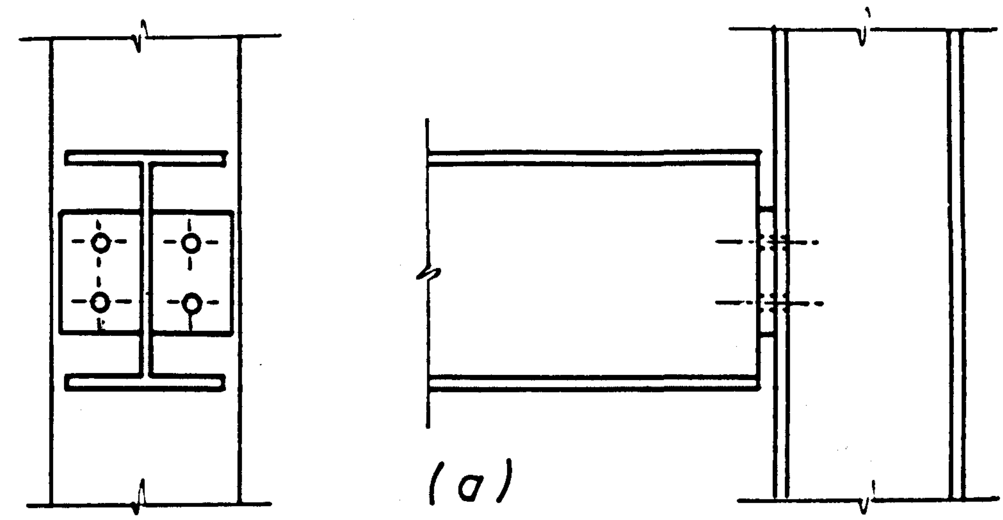
Reference of experimental data	Date	Country of origin	Number of tests	Type of fastener	Test type	Comments on M-Φ curves
Sommer	1969	Canada	20	¾" A325 bolts	Cantilever	M-Φ curves available for each test
Bennetts et al	1978	Australia	2	M20 Gr 8.8 bolts	Propped cantilever	M-Φ curves available for each test
Pham & Mansell	1982	Australia	7	M20 Gr 8.8 bolts	Propped cantilever	Only shear - Φ, shear deflection curves provided
Hafez	1982	Canada	8	¾" A325 bolts	Cantilever	M-Φ curves available for each test
Zandonini & Zanon	1986	Italy	3	M20 Gr 4.8 bolts and M16 Gr 6.8 bolts	Cantilever	M-Φ curves available for for each test
MacIntyre	1988	Canada	3	¾in. A325 bolts	Cantilever	M-Φ curves available for each test

Table 2-2, Available experimental data for flush end plate connections (after Benterkia(1991))

Reference of experimental data	Date	Country of origin	Number of tests	Type of fastener	Test type	Comments on M-Φ curves
Ostrander	1970	U.S.A.	13 unstiffened + 11 stiffened	Cruciform load in column	[illegible] in diam A325 bolts	Provided for each test
Zoetemeijer	1974	Netherlands	2 unstiffened	Cantilever	M20 & M22 G10.9 bolts	Provided for each test
Zoetemeijer	1974	Netherlands	4 unstiffened	Portal frame	M22 Gr Gr 10.9 bolts	Provided for each test
Zoetemeijer & Kolstein	1975	Netherlands	8 unstiffened + 4 stiffened	Cruciform load in column	M24 Gr 8.8 bolts	Provided for each test
Zoetemeijer	1981	Netherlands	23 unstiffened	Cantilever	M24 Gr 8.8 Bolts	Provided for 6 tests + 7 supplied privately
Phillips & Packer	1981	Canada	5 stiffened	Cantilever	[illegible] in diam A325 bolts	Provided for each test
Bose	1981	U.K.	1 unstiffened	Cruciform load in column	M20 Gr 8.8 bolts	Provided for each test
Morris & Newsome	1981	U.K.	4 stiffened	Cruciform	[illegible] in diam bolts	Provided for each test
Man & Morris	1981	U.K.	6 stiffened	Cruciform load in column	M22 HSFG bolts	Only moment-deflection curves provided
Tong	1985	U.K.	6 stiffened	Cruciform load in column	M20 Gr 8.8 bolts	Provided for each test
Zandonini & Zanon	1986	Italy	3 unstiffened	Cantilever	M20 Gr 4.8 + M16 Gr 6.8 bolts	Provided for each test
Davison	1987	U.K.	1 unstiffened	Cruciform load in bolts	M16 Gr 4.6	Provided for each test
Prescot	1987	U.K.	6 stiffened	Cruciform load in column	M20 Gr 8.8	Provided for each test
Chakrabati	1987	U.K.	2 unstiffened	Cruciform load in column	M16 Gr 8.8 Bolts	Provided for each test

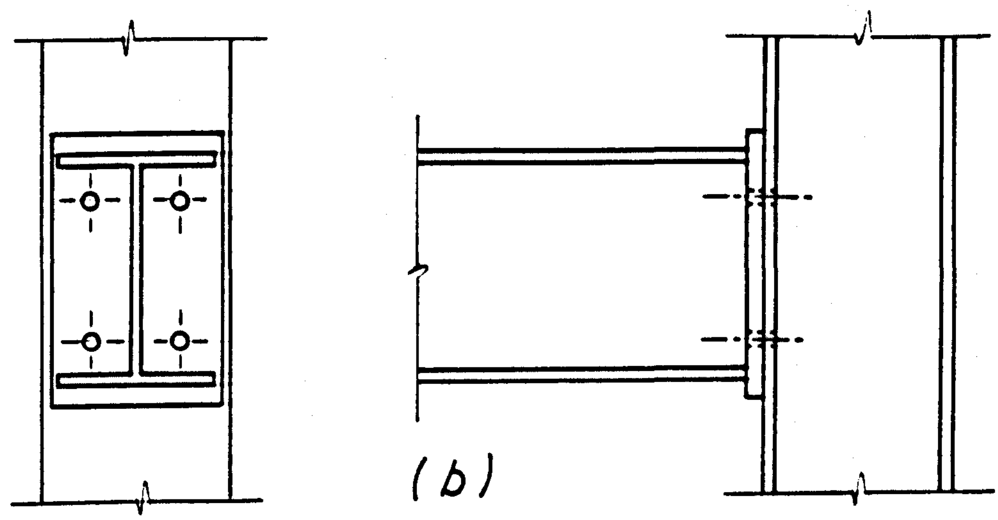
Table 2-3, Available experimental data for extended end plate connections (after Benterkia(1991))

Reference of experimental data	Date	Country of origin	Number of tests	Type of fastener	Test type	Comments on M-Φ curves
Johnson et al	1960	U.K.	1 stiffened	Cruciform	[illegible] in HT bolts	Provided for each test.
Sherbourne	1961	U.K.	1 unstiffened 4 stiffened	Cruciform	[illegible] & [illegible] in HT bolts	Provided for each test.
Mann	1968	U.K.	5 unstiffened stiffened	Cruciform	[illegible], 1 & 1.1/8 in HSFG bolts	Provided for each in test.
Bailey	1970	U.K.	3 unstiffened 10 stiffened	Cruciform	HSFG bolts	Provided for each test.
Zoetemeijer	1974	Netherland	8 unstiffened	4 Cantilever 4 Portal frame	M20 & M22 Gr 10.9 bolts	Provided for 4 tests only.
Packer	1977	U.K.	4 ustiffened 1 stiffened column	Cruciform load in	M15 HSFG bolts	Provided for each test.
Ioannides	1978	U.S.A.	6 unstiffened	Cruciform	[illegible], [illegible] & 1 in A325 bolts	Provided for each test.
Dews	1979	U.S.A.	3 unstiffened	Cruciform	[illegible] & 1 in A325 bolts	Provided for each test
Grundy et al	1980	Australia	2 unstiffened	Cruciform load in column	[illegible] HT bolts	Provided for each test.
Johnstone & Walpole	1981	New Zealand	8 unstiffened	-	M30 & M24 Gr. 8.8 bolts	Provided for each test.
Tarpy & Cardinal	1981	U.S.A.	14 unstiffened 2 stiffened	Cruciform load in column	[illegible], [illegible] & 1 in A325 bolts	Provided for 4 tests.
Graham	1981	U.K.	21 unstiffened	Crucıform load in column	M16 HSFG bolts	Provided for each test.
Moore & Sims	1983	U.K.	2 unstiffened 2 stiffened	Cruciform	M16 Gr 8.8 bolts	Provided for two only.
Zoetemiejer & Munter	1984	Netherland	4 unstiffened	Cruciform	M20 Gr 8.8 bolts	Provided for each test.
Tong	1985	U.K.	1 unstiffened 11 unstiffened	Cruciform	M20 Gr 8.8 bolts	Provided for each test.
Zandonini & Zanon	1986	Italy	10 stiffened	Cantilever	M20 Gr 8.8 bolts	Provided for each test.
Prescott	1987	U.K.	4 stiffened	Cruciform	M20 Gr 8.8 bolts	Provided for each test.
Davison	1987	U.K.	4 stiffened	Cruciform load in column	M16 Gr 8.8 & HSFG	Provided for each test.
Chakrabarti	1987	U.K.	1 unstiffened	Cruciform	M16 Gr 8.8 bolts	Provided for each test.
Janss et al	1987	Belgium	6 unstiffened	Cantilever	Grade 10.9 H.S. bolts	Provided for each test.



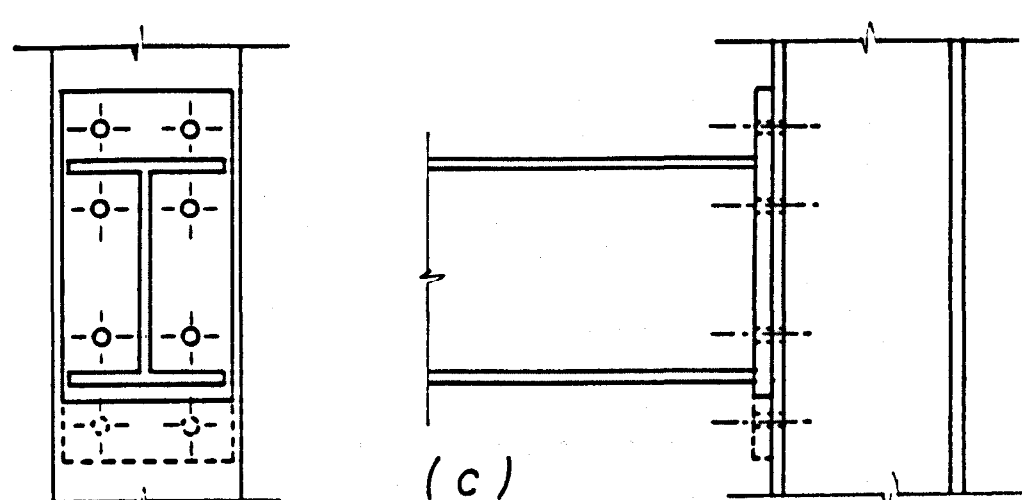
(a)

Header plate connection



(b)

Flush end plate connection



(c)

Extended end plate connection

Fig. 2-1, End plate connections

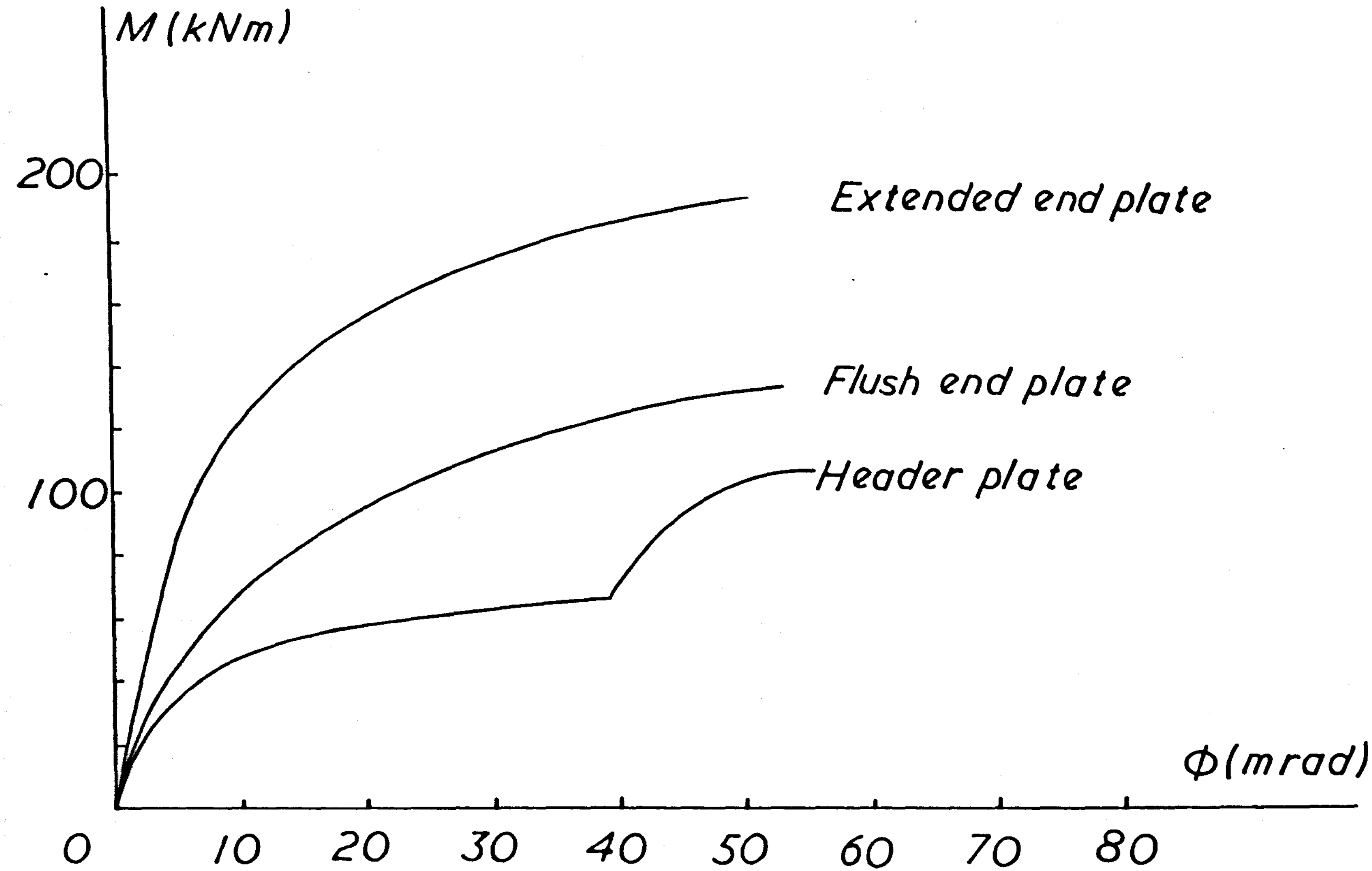


Fig. 2-2, Typical moment-rotation curves of end plate connections

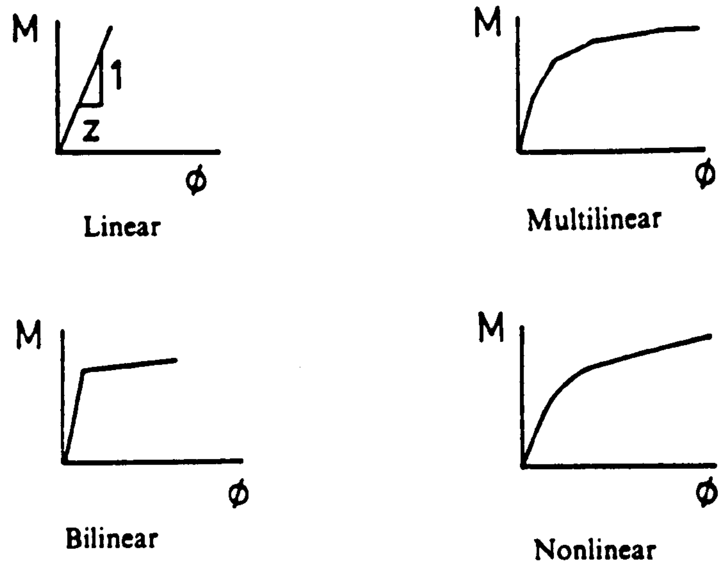


Fig. 2-3, Representation of moment-rotation behaviour of connections

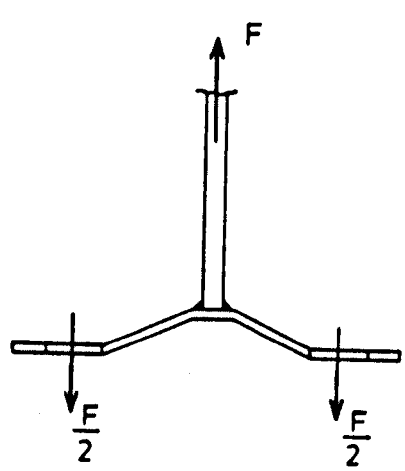


Fig. 2-4, T-stub model used in analysis of header plate connections (Kennedy & Hafez(1984))

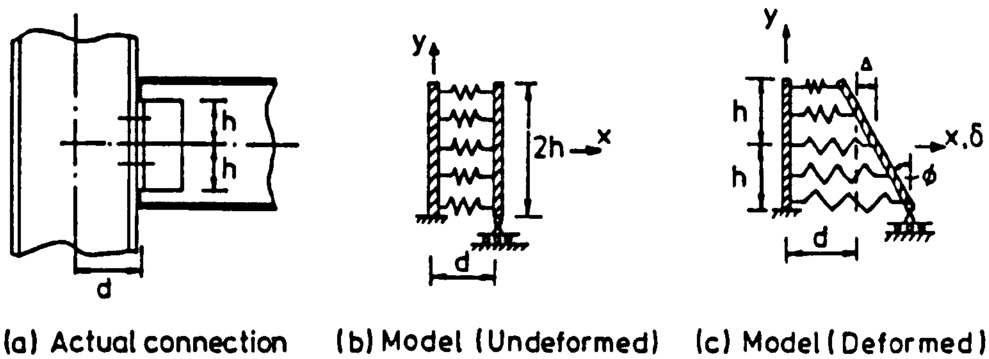


Fig. 2-5, Mechanical model of web cleat connections (Wales & Rossow(1983))

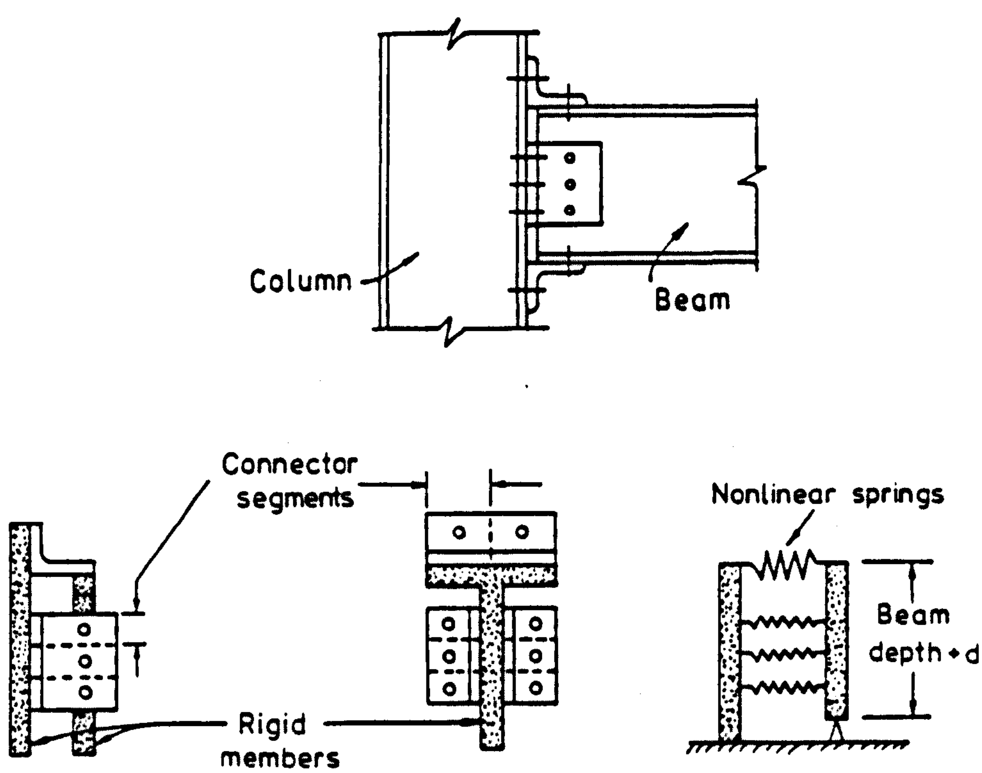


Fig. 2-6, Mechanical model for a flange and web cleated connection (Chmielowiec & Richard(1987))

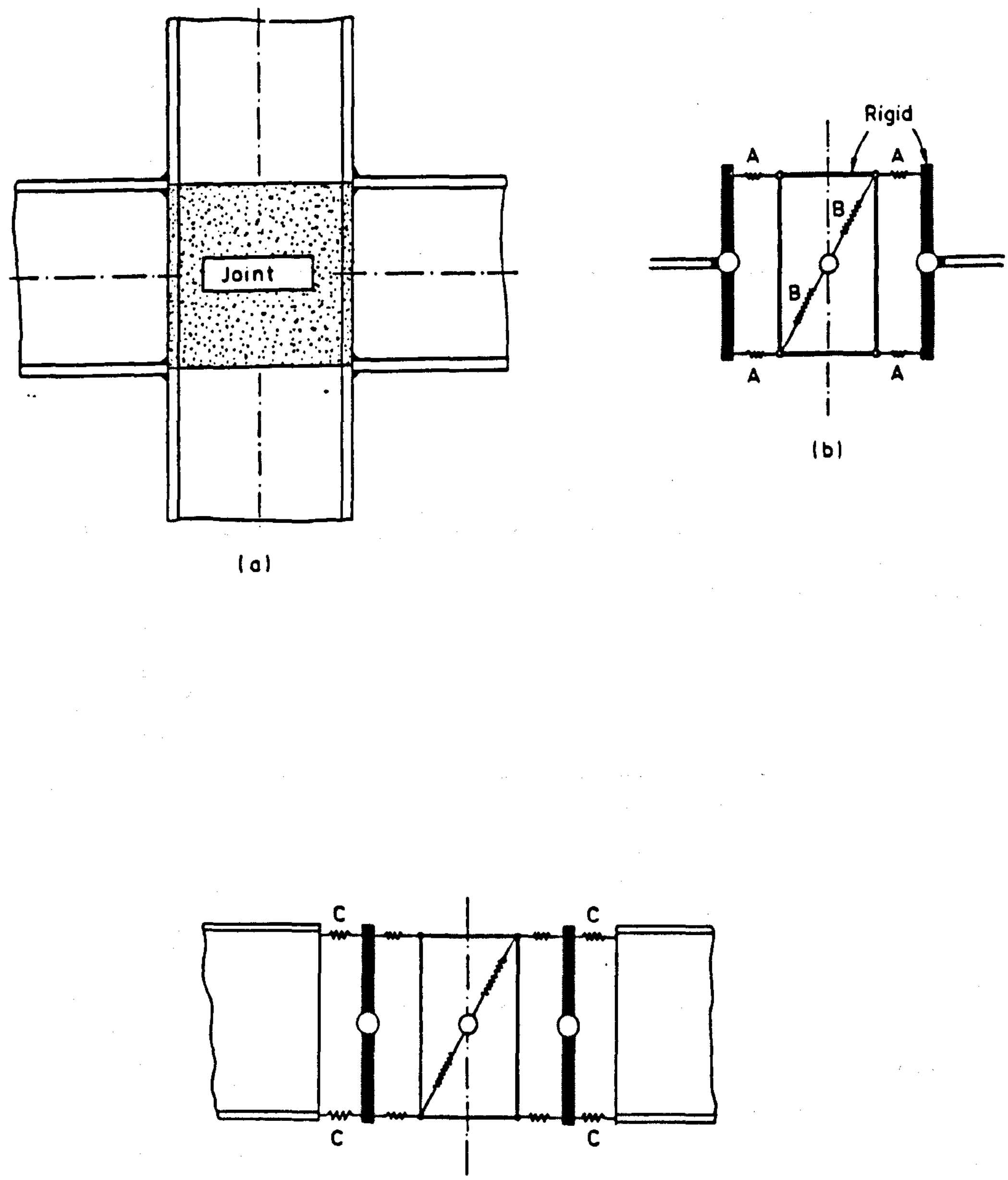


Fig. 2-7, Mechanical model for welded and bolted connections (Tschemmernegg(1988))

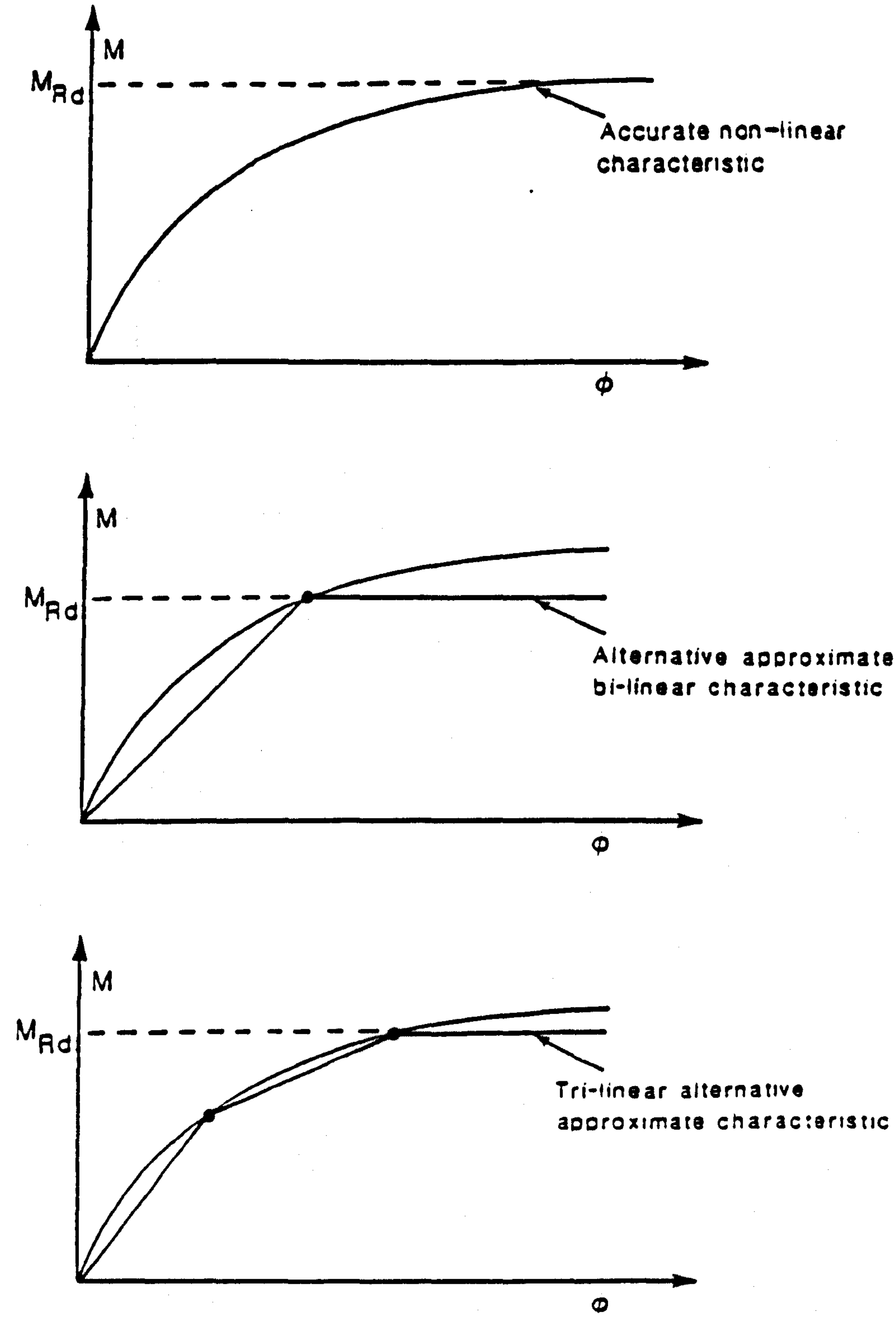
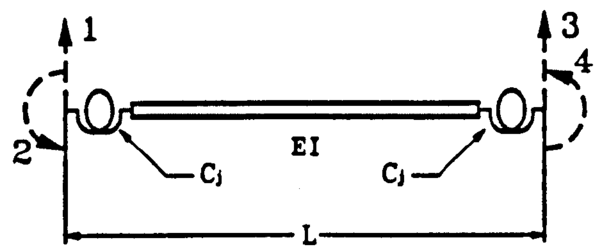


Fig. 2-8, Derivation of approximate moment-rotation characteristics (EC3)



$$K_{ij}^{SR} = \frac{1}{A} S_{ij} \cdot K_{ij}^{R}$$

$$K_{ij} = \text{element stiffnesses} \begin{cases} K^R \text{ rigid restraint} \\ K^{SR} \text{ semi-rigid restraint} \end{cases}$$

$$S_{11} = S_{33} = (1+\alpha)$$

$$S_{22} = S_{44} = (1+\frac{3}{2}\alpha)$$

$$S_{24} = 1$$

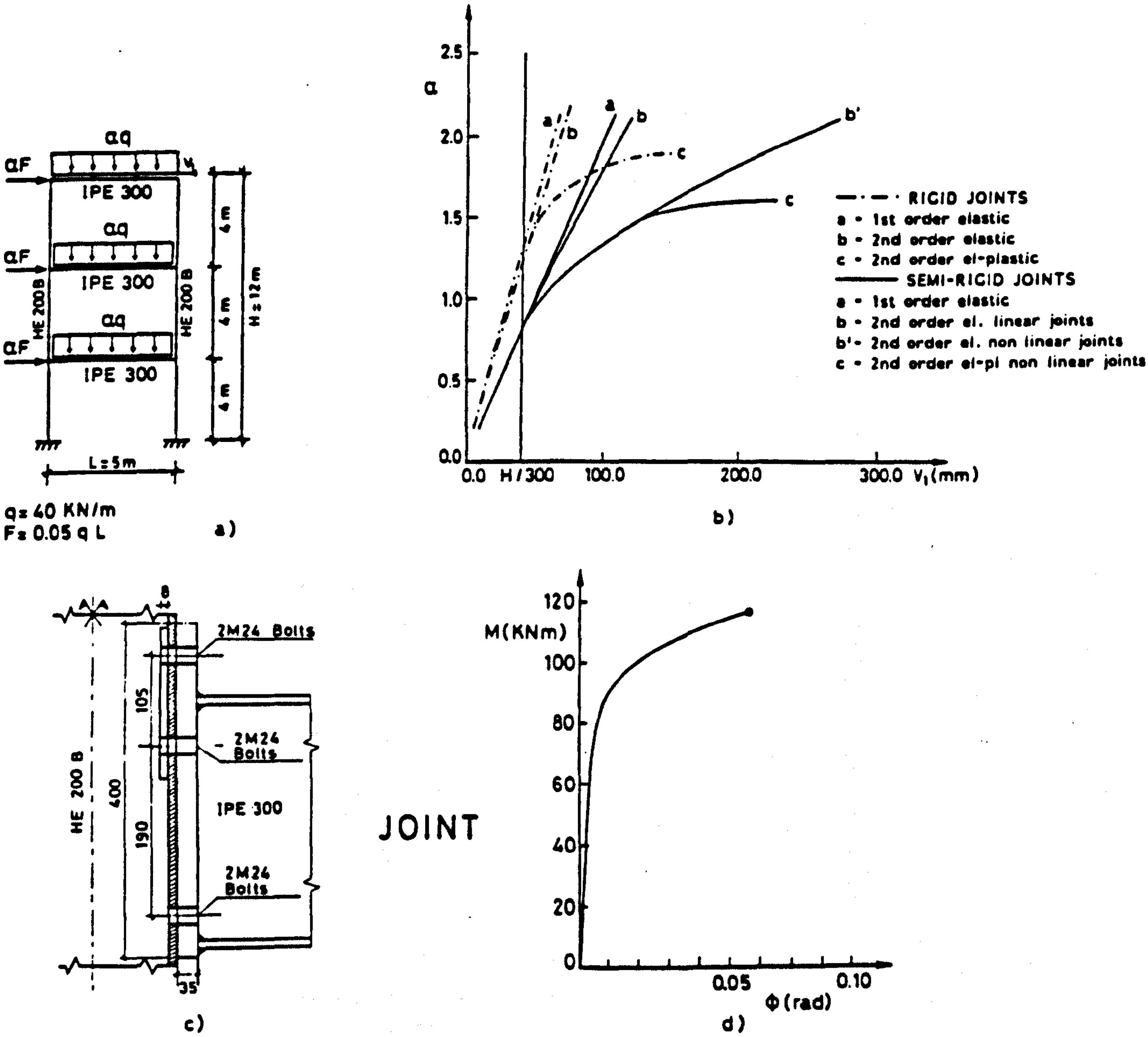
$$\left.\begin{matrix} S_{12} = S_{34} \\ S_{31} = S_{41} \\ S_{32} \end{matrix}\right\} = (1+\alpha)$$

$$S_{ij} = S_{ij}$$

$$\alpha = \frac{2EI_b}{C_j L}$$

$$A = 1+4\alpha+3\alpha^2$$

Fig. 2-9, Correction coefficients for member stiffness



(a) The frame
(b) The responses
(c) The joint
(d) The M-φ curve of the joint

Fig. 2-10, Influence of joint flexibility on the response of an unbraced frame

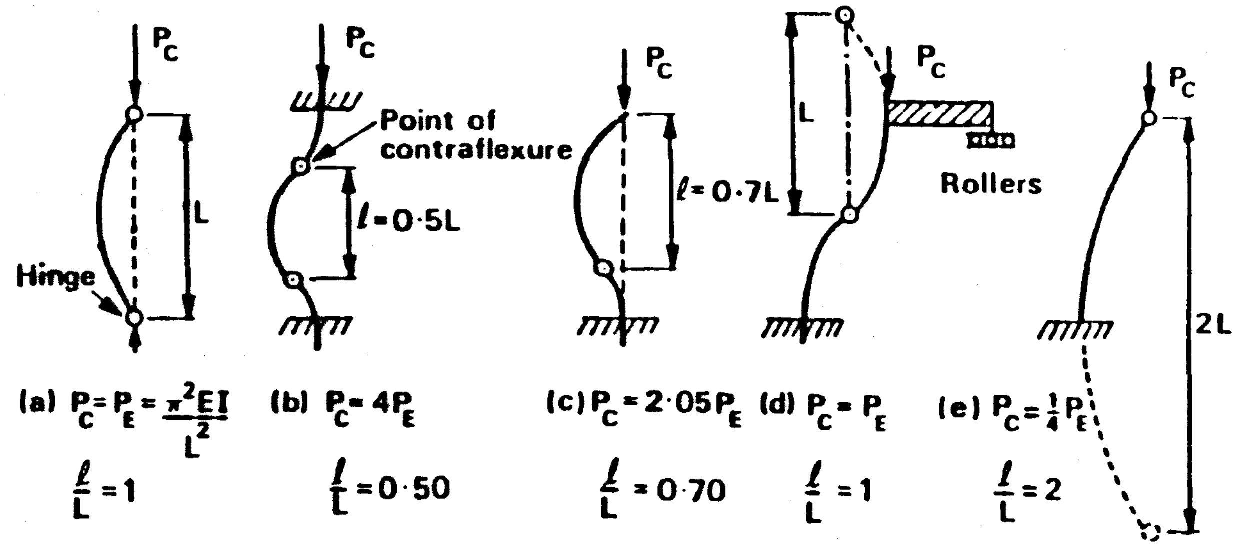
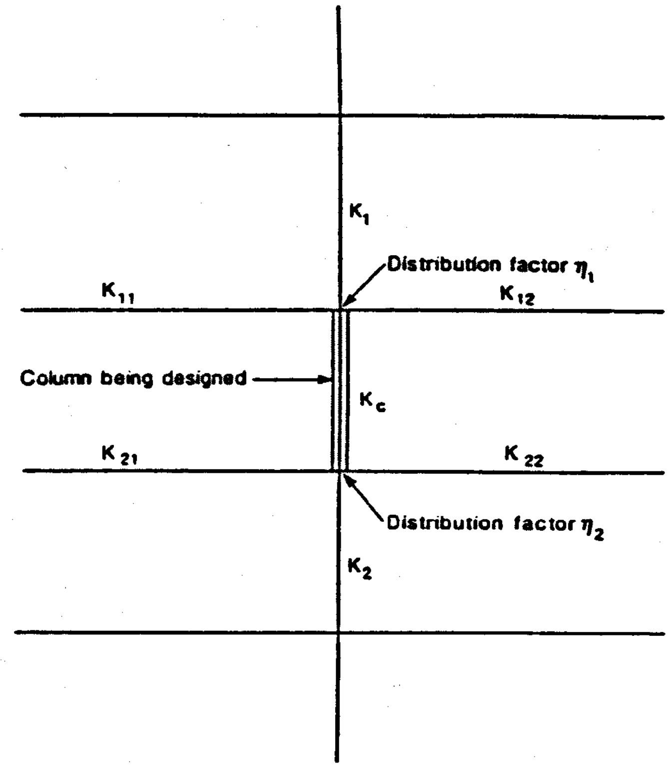
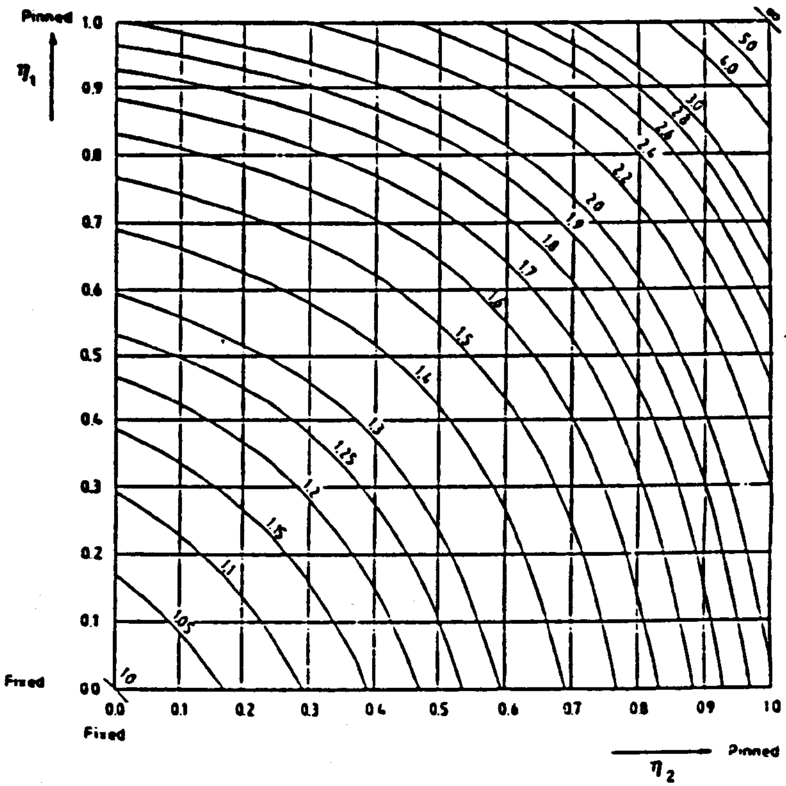


Fig. 2-11, Idealized solutions for critical loads and effective lengths



$$\eta_1 = \frac{K_c + K_1}{K_c + K_1 + K_{11} + K_{12}}$$

$$\eta_2 = \frac{K_c + K_2}{K_c + K_2 + K_{21} + K_{22}}$$



(a) Distribution factors for continuous columns

(b) Buckling length ratio l/L for a column in a sway mode

Fig. 2-12, Determination of effective length of column to EC3

Chapter 3
STUDIES ON UNBRACED STEEL FRAMES WITH SEMI-RIGID CONNECTIONS

This chapter deals with different aspects of connection flexibility, in particular, its effect on sidesway in unbraced steel frames at serviceability, and on the collapse load at the ultimate state.

The Eurocode 3 classification of connections is first examined by analysing a number of frames with semi-rigid connections whose behaviour is that of the boundary between rigid and semi-rigid as defined in EC3. The sway responses of these frames are determined and compared with those obtained from frames with extended end plate connections.

The polynomial moment-rotation curve for end plate connections proposed by Frye & Morris(1975) is used in the analysis of a number of frames. The resulting sways are compared with those obtained by using the EC3 formula for prediction of connection stiffness.

The "wind-moment" or "wind-connection" method is used to design two basic frames. They are analysed under variable wind loads assuming that the frames are first rigid and then semi-rigid with the connection behaviour given by Frye & Morris formula. The resulting horizontal deflections are compared and the effect of variation in wind load is indicated.

To choose the most convenient procedure in design, it is helpful to know at an early stage whether the "ultimate strength" or the "serviceability limit" dominates. Thus, a quick estimation of horizontal deflections is needed. The "desk method" for estimating the sway response of frames proposed first by Wood(1974) is examined. Semi-rigid joints are included in the method, which is then applied to several frames with extended end plate connections. The results are studied to check the accuracy of the method.

In the preceding studies, to be reported in this chapter, the frames were considered in the serviceability state. In the last section of this chapter, the ultimate state of frames is investigated. The $P-\Delta$ effects which may be less influential in serviceability, may become a significant factor at the ultimate state. Furthermore, elastic analysis cannot be applied when the formation of plastic hinges is considered. A second-order elastic-plastic computer program is then used to analyse a basic frame with full and partial strength connections. The influence of semi-rigid connections on the internal moments of frame at collapse is investigated for these two types of connection.

3-1-Study on EC3 Classification of Connections

The classification of connections explained in Chapter 1 has been adopted by EC3(1992). In this code, the beam-to-column connections are classified by strength or by rigidity. The classification by strength divides the connections into nominally pinned, full-strength and partial strength with respect to the design moment resistance of the beam. The classification by rigidity differentiates between three types of connections; namely nominally pinned, rigid and semi-rigid.

The limiting behaviour for a nominally rigid beam-to-column connection is represented by a tri-linear moment-rotation characteristic, defining the moment resistance and the rotational stiffness. The EC3 classification boundaries for beam-to-column connections in braced and unbraced frames are shown in Fig. 3-1.

If the moment-rotation characteristic of a beam-to-column connection lies above the solid line on the appropriate diagram in Fig. 3-1, it will be considered as rigid; similarly, if it lies below the line, it is defined as semi-rigid.

A verification seemed to be needed for this definition. It appears of particular importance when the sway limitation for design of unbraced frames is considered. It has been reported in Chapter 2 that semi-rigid connections increase significantly the sway deflections in such frames due to the overall flexibility given to the frame. Consequently, connections with a moment-rotation characteristic coincident with the boundary line in the EC3 definition, will increase the sway deflection when compared to the similar fully-rigid frame. This additional sway will be neglected in design as the connections can be treated as rigid.

The investigation was carried out by designing several fixed-based frames with different geometries. "Exact" analysis then was undertaken to determine the increase of sway due to the flexibility of nominally rigid connections. The moment-rotation characteristic was taken as the initial stiffness given by the boundary line in Fig. 3-1(a). Two computer programs were used for design and analysis.

3-1-1-Design of Frames

The "wind-connection" method was used to design the frames. This approach has been described in Chapter 1.

A program for design using British sections and end plate connections has been written by Reading(1989). In this program, the geometry of the plane frame, the type of connections, the grades of steel for members and connections, the wind speed and the vertical loads are given as input data. The program calculates the wind loads for each storey and combines them with gravity loads in accordance with BS5950:Pt 1. The program generates the required joints, and the beam and column sections.

All the frames were designed by applying wind loads in the plane of the frame, bending columns about their strong axes. The wind speed was taken as 45 m/s and typical gravity loads for office accommodation were assumed.

3-1-2-Analysis of Frames

The analysis was undertaken by the use of an elastic semi-rigid analysis program developed by Benterkia(1991). It was used to obtain the exact forces and moments in the members and displacements at the joints. The second-order effects were taken into account, but usually the $P-\Delta$ effect would be significant only at failure. So this effect was unlikely to affect appreciably the results at serviceability. The analysis described is known as "rigorous analysis". In this program the following input data is given: The section area and the second moment of area for each member, the geometry of frame, the applied loads and the moment-rotation characteristics of the semi-rigid joints. From the input data the program forms the compact stiffness matrix and gives the joints displacements and rotations, and members' forces and moments.

For comparison between the horizontal displacements, only the unfactored wind loads were applied. Initially, it was assumed that the connections were rigid and the analysis was carried out to obtain the displacements. Then it was assumed that the connections were semi-rigid with the connection stiffness at the limit given in EC3 for classification as rigid. The relationship:

$$\bar{m} = 25\,\bar{\phi} \tag{3.1}$$

was used which gives the relevant stiffness relationship as:

$$C = \frac{25EI_b}{L_b} \tag{3.2}$$

where C is the connection stiffness,

E is the modulus of elasticity assumed as 20500 kN/cm^2,

I_b is the beam second moment of area, and

L_b is the length of the beam.

The use of the initial linear part of limiting stiffness for rigid connections would seem reasonable at serviceability limit state particularly if the attention be paid to the procedure of loading and unloading the connection. Even if the connection had previously been loaded to a moment level $\bar{m}$ greater than 0.67, e.g. by the action of excessive gravity loads, moment reversal would take place along a linear path, of slope similar to the initial stiffness of the loading curve. Thereafter, the connection response to load variations at the working level will proceed elastically. Furthermore, previous investigations (Moncarz & Gerstle(1981)) have shown that, the assumption of linear connection behaviour leads to elastic design moments which are very close to the more exact values based on non-linear connection behaviour. This simplification appears conservatively acceptable if the linearization lies inside the nonlinear curve instead of being externally tangent to the curve (see Fig. 3-2). The former is suggested by EC3 as the secant stiffness C in bi-linear and tri-linear moment-rotation presentation.

Initially, two frames as shown in Fig. 3-3 were investigated. Comparison of the results provided an indication of the sensitivity of side-sway to joint stiffness and to the geometry of frame. The recorded increase of sway in Fig. 3-3 is that due to the flexibility of nominally rigid connections compared to connections taken as fully rigid. The horizontal deflection at the top of the structure has been compared to find the percentage increase of sway.

At the next stage, a number of frames with different dimensions, as shown in Table 3-1, were studied in order to investigate the effect of different parameters on the increase of sway due to semi-rigid behaviour of nominally rigid connections.

3-1-3-Parametric Study

The following parameters influenced appreciably the magnitude of sway increase:

a) overall height to width ratio for the frame,

b) sizes of beam section,

c) sizes of column section,

d) grades of steel.

Each of these is explained in detail in the following sub-sections.

3-1-3-1-Overall Height to Width Ratio for the Frame

Reference to Table 3-1 'shows generally a significant increase in sway when the configuration of the structure varies from 2 storey,4 bay to 8 storey,1 bay. The structural steel has been assumed as Grade 50. The height increase has two different influences, increasing both wind loads and slenderness.

By comparing the 6 storey,2 bay frame with the 6 storey, 1 bay structure, it is seen that less increase occurs in the former. This is because two beams are available to restrain three columns, compared with only one beam restraining two columns in the single bay frame. The total beam stiffness relative to the total column stiffness is therefore less in the single bay frame. Hence the beam stiffness is of a more significant effect to the response of the single storey structure. It follows that an effective reduction in this stiffness due to connection flexibility is more noticeable in a single storey frame.

A similar effect is observed in the 8 storey frames, as the bay width increases. Again, the beam effect becomes more significant as its flexibility increases due to the longer span. Further reduction in the beam stiffness due to connection flexibility becomes more noticeable in the wider frames.

Considering the influence of number of storeys, two factors need to be considered:

1) The overall sway of a frame with few storeys is more influenced by the base fixity, which remained fully rigid in the comparisons.

2) The general tendency is for the total beam stiffness to reduce, relative to that of the columns, as the number of storeys increases. This is because the column sections are strongly influenced by the wind moments, whose maximum magnitude increases

with the number of storeys. Once more therefore, the beams and the reduction in the beams' stiffness due to connection flexibility become more influential in the frames of more storeys.

3-1-3-2-Sizes of Beam Section

Reference should be made to Fig. 3-1, which defines the limiting moment-rotation characteristics of beam-to-column connections. Since the use of Eqns. (3.1) and (3.2) directly relates the characteristic of a connection to the beam stiffness, the sway will be significantly affected by the section size of the beam.

Researchers including Ackroyd(1987) have investigated the wind-connection method. This method is believed to overestimate the beams, since they are assumed simply supported, and to underestimate the columns. Gerstle(1985) had pointed out that for the frames with more stiffness, that overestimation is more significant than this underestimation, specially for interior columns. Regarding this fact, the minimum possible beam sections were assigned by implying negative moments at restraints and checking the deflection limit in accordance with Cl. 2.5 of BS5950:Pt 1. The results are summarized in Table 3-2.

The reason for the changes in increase of sway shown in Table 3-2 is the same as that given in 3-1-3-1. By reducing the beam sections, the side-sway increases. Reduction in the effective stiffness of the beams due to connection flexibility is more noticeable when the beam sections are of smaller size.

3-1-3-3-Sizes of Column Section

It is common in design of steel structures to adopt the same size of columns for each rise of two storeys or more, because of the fabrication cost. In Table 3-1, the frames maintain the size of column section over at least two storeys. If the exact calculated sizes for column sections are used, i.e. less stiff columns, a significant reduction occurs in the sway increase. For instance the result is given for an 8storey,1bay frame in Table 3-3. This is because the less stiff columns have a more significant effect on the sway deflection, relative to the effect of the connections.

3-1-3-4-Grades of Steel

All the connections were assumed to be extended end plates of Grade 43 steel with an unstiffened column flange and web. None of the details of the connections was changed in the comparisons.

The grade of steel of beams and columns has been changed by assuming in turn grades 43 , 50 and 55. The results for two types of frames with different overall stiffnesses are given in Table 3-4. Generally, using a low grade of steel would result in less increase in sway, because of the greater stiffness given to the beams. To compute the effect of changing to a higher grade of steel, Grade 55 was specified for 8storey,1bay frame; it is recognized though that this would not be used in practice.

3-1-4-Conclusion

Extended end plates bolted to the stiffened columns had been found to cause increases of sway in the range of 25-50% as reported by Anderson et al(1991). This is compared with a maximum of 15% obtained by using the EC3 boundary for rigid connections in the frames reported here. Thus, the EC3 definition for rigid joints in unbraced frames exceeds the rigidity of stiffened extended end plate which is recognized as a very stiff connection. Only the moment-rotation curves of T-stubs and fully welded connections would lie in the rigid region of EC3 classification (see Fig. 2-2). However these joints are not in common use in the building industry. Thus, all commonly used connections may be classified as semi-rigid according to EC3, if used in unbraced frames.

Having considered the degree of stiffness of the boundary line in the EC3 classification of connections, it can be used as an upper limit to the connection rigidity. Therefore, any result from the sway analysis of frames with this stiffness can be considered as the lower limit to the sway deflection. In the other words, the sway limitation in the current codes of practice may be modified if the effect of such a stiff connection on the sway of frames is recognised.

In BS5950 and in ECCS Recommendations(1978), the serviceability limitation of horizontal deflection in each storey is given as the ratio "height/300". This author argues that this recommendation is conservative if a semi-rigid analysis is made of a frame. As it was shown in Table 3-1, an increase of about ten percent in side-sway is found for frames with nominally rigid connections if a semi-rigid analysis is made. This increase will be ignored if a rigid analysis is permitted. Accordingly, an allowable sway ratio of 1/270 for unbraced medium rise frames is more realistic. In EC3, the overall limit is 1/500. For the same reason this should be relaxed to 1/450 for semi-rigid analysis.

3-2-Study on Sway of Frames with Semi-Rigid Connections

To calculate the sway deflections of semi-rigid frames, the actual $M-\phi$ relationships of the connections are required. As described in Chapter 2, several prediction equations have been derived by researchers. The author used equations due to Frye & Morris(1975) and also the method given by EC3. An evaluation of the former and a comparison between the two methods is then possible.

These two methods are employed in the sway calculation of three basic frames. The effect of semi-rigid connections on the sidesway of these frames is considered. Comparison is made between the results using the above methods. A criteria for design at serviceability limit state is proposed, based on the results of the analyses.

3-2-1-Frye & Morris Relationship

Practical prediction equations for moment-rotation characteristics of connections were developed by Frye and Morris in 1975. The following assumptions were made in their study:

a) The effects of shear and axial load on the connection's deformations may be ignored.

b) The members connected at the semi-rigid joint are prismatic and straight.

c) Possible buckling of individual members or portions of the structure is ignored.

d) The material of components is linearly elastic and the effects of strain hardening is neglected.

In order to incorporate the moment-rotation relationship into a structural analysis program, a standardization procedure was carried out by Frye and Morris on the available experimental data. The general form of the M–ϕ relationship was then suggested (for seven types of connections) as:

$$\phi = C_1(KM)+C_2(KM)^3+C_3(KM)^5 \tag{3.3}$$

where C_i are constants found by curve fitting, and

K is the standardization factor found by consideration of a family of experimental M–ϕ curves. K depends on the main geometrical parameters of the particular connection under consideration and is assumed to have the form:

$$K = \prod_{j=1}^{j=m} P_j^{a_j} \tag{3.4}$$

where P_j = the numerical value of j^{th} size parameter,

a_j = dimensionless exponent which indicates the effect of j^{th} size parameter on the moment-rotation relationship, and

m = total number of size parameters.

3-2-1-1-End Plate Connection

The proposed formula for an end plate connection with column stiffeners is given by the above researchers as:

$$\phi = 1.79(KM)10^{-3}+1.76(KM)^3 10^{-4}+2.04(KM)^5 10^{-4} \quad (3.5)$$

where $K = d^{-2.4}t^{-0.6}$ based on experimental research by Ostrander(1970) and Sherbourne(1961).

A similar formula for an end-plate connection without column stiffeners is given based on test results obtained by Johnston et al(1960), Ostrander and Sherbourne:

$$\phi = 1.83(KM)10^{-3}-1.04(KM)^3 10^{-4}+6.38(KM)^5 10^{-6} \quad (3.6)$$

where $K = d^{-2.4}t^{-0.4}f^{-1.5}$, and: d is the distance between two extreme end rows of bolts,
t is the end-plate thickness,
f is the column flange thickness.

In the above equations M is in kips; ϕ in radians; d , t and f are in inches. No distinction is made between a joint using flush end plate and one with an extended end plate.

3-2-1-2-Modified Relationships

The above relationships have been recently converted to metric units by Benterkia(1991) and the following formulae have been given. For an end plate connection with stiffeners:

$$\phi = 1.79(1.450KM)10^{-3}+1.76(1.450KM)^3 10^{-4}+2.04(1.450KM)^5 10^{-4} \quad (3.7)$$

where $K = d^{-2.4}t^{-0.6}$.

For an end plate connection without stiffeners:

$$\phi = 1.83(4.873KM)10^{-3}-1.04(4.873KM)^3 10^{-4}+6.38(4.873KM)^5 10^{-6} \quad (3.8)$$

where $K = d^{-2.4} t^{-0.4} f^{-1.5}$. In these equations M is in kNcm, ϕ in radians; d, t and f in cm.

Furthermore, a suggestion has been made by Goverdhan(1984) that d be taken as the depth of the beam instead of the distance between the extreme bolt rows. Recently, Benterkia modified the power of d in the above equations from -2.4 to -2.6. He then reported a better approximation in comparison with the available test data, assuming d as the beam depth. He proposed that for extended end plate joints, the formula for an unstiffened connection is the most accurate among the available methods, and that for a stiffened connection is reasonably accurate, if the above modifications are made, but suggested that the Frye & Morris formulae should not be used at all for flush end plate connections.

In the analyses by the author, Eqn. (3.8) has been used. The value of d has been taken as the beam depth plus the distance from the centre of the bolt in the extended part of end plate to the top of the top beam flange. This is closer to the original Frye & Morris assumption. Frye and Morris had assumed that the end plate is extended both in tension and compression zone, while the author has assumed that the end plate is extended only in the tension zone.

3-2-2-Eurocode 3 Relationship

In Annex J of EC3, the moment-rotation characteristic of a connection is determined by the moment resistance and the rotational stiffness. The rotational stiffness of a bolted end plate beam-to-column connection is approximated by:

$$C = \frac{E h^2 t_{wc}}{\sum \frac{\mu_i}{k_i} \left(\frac{F_i}{F_{i.Rd}}\right)^2} \tag{3.9}$$

where

C is the secant stiffness with respect to a particular moment M in the connection (less than the design moment resistance of the connection),

E is the Young's Modulus,

h_1 is the distance from first row of bolts below tension flange of the beam to the centre of reaction of compression zone,

μ_i is the modification factor for component i,

k_i is the stiffness factor for component i,

F_i is the force in component i of the connection due to the moment M, but not less than $\frac{2}{3}$ of its design resistance force.

3-2-2-1-The Stiffness Factor

The stiffness factor for each component is specified separately, and for an unstiffened connection should be taken as follows:

Column web , shear zone	$k_1 = 0.24$
Column web , tension zone	$k_2 = 0.80$
Column web , compression zone	$k_3 = 0.80$
Column flange , tension zone	$k_4 = \frac{t_{fc}^3}{4m^2 t_{wc}}$
Bolts , tension zone	$k_5 = \frac{2A_s}{l_b t_{wc}}$
End-plate , tension zone	$k_6 = \frac{t_e^3}{12\lambda_2 m^2 t_{wc}}$ but $\geq \frac{t_e^3}{4m^2 t_{wc}}$

where t_{fc} is the column flange thickness,

t_{wc} is the column web thickness,

t_e is the end-plate thickness,

A_s is the cross section area of each bolt,

m, l_b and λ_2 are defined in Fig. 3-4.

For any stiffened component, the relevant stiffness factor should be taken as infinity except where the column has a stiffener in the tension zone. In that case:

$$k_4 = \frac{t_{fc}^3}{12\lambda_2 m^2 t_{wc}} \quad but \quad \geq \frac{t_{fc}^3}{4m^2 t_{wc}}$$

3-2-2-2-The Modification Factor

The modification factor μ_i for i=1,2,3 should be taken as 1, and for i=4,5,6 obtained from:

$$\mu_i = \frac{h_1 F_{1.Rd}}{M_{Rd}}$$

where $F_{1.Rd}$ is the force in the first row of bolts below the tension flange of the beam, due to the moment M_{Rd}.

M_{Rd} is the design moment resistance of the connection.

In the case of an extended end plate connection, the stiffness provided by the extended part of end plate should be taken into account. The correspondent connection stiffness C is the larger value of the two calculated stiffnesses, one based on the extended part and the other by omitting it. For the extended part, h_1 should be replaced by h_e which is the distance from the bolt row in the extended part to the centre of reaction of the compression zone. The stiffness factor k_6 should then be taken as:

$$k_6 = \frac{t_e^3}{4m_x^2 t_{wc}}$$

where m_x is defined in Fig. 3-4.

3-2-3-Comparison of Sways Using Two Groups of Formulae

In order to compare the sways obtained by the use of the two different prediction equations, namely EC3 and Frye & Morris, three basic frames with different overall stiffness were examined:

a) Eight storey, two bay.

b) Six storey, two bay.

c) Four storey, two bay.

Extended end plate connections without stiffeners were used and the frames had practical dimensions. They were designed by the "wind-moment" method in conjunction with BS5950:Pt 1. The latter permits Dead+0.8(Imposed+Wind) to be taken as the combined loading in which the maximum wind load and minimum gravity loads were used. This combination of loading gives more sway deflections than minimum wind and maximum vertical loads (Reading(1989)). Further information about the geometry and the loading of frames can be obtained from Fig. 3-5.

3-2-3-1-Moment-Rotation Data

The moment-rotation characteristics of the connections were obtained by means of two computer programs. The design program written by Reading(1989) provides stiffness-rotation data according to the EC3 formula, in the form of a multilinear representation as shown in Fig. 3-6.

A program was written by the author to provide moment-rotation (or stiffness-rotation) data in accordance with the Frye & Morris relationships. The values of the three variables, i.e. beam depth, end-plate thickness and column flange thickness are tabulated for the basic frames in Table 3-5.

3-2-3-2-Results

The frames were analysed by means of an elastic-plastic computer program developed by Kavianpour(1990) that could deal with both rigid and semi-rigid connections. The sway deflections which resulted from the EC3 method and the Frye & Morris equations are compared in Table 3-6 together with those from rigid analysis.

It can be seen from Table 3-6 that the Frye & Morris prediction equation causes an increase of sway deflections in semi-rigid frames from 30% to 53% over rigid frames, while EC3 equation results in much greater sways ranging from 122% to 156%. The difference between these two groups of results appears to be less when the frames change from low-rise to medium-rise.

3-2-4-Comparison Between Frye & Morris and EC3 Equations

Frye and Morris have included three parameters as the principal components of end plate connections. In reality, the connection behaviour is affected by several more parameters including the strength of individual components and the behaviour of fasteners. The EC3 formula includes more parameters, for example the effects of the column web in shear, tension and compression have been included in the EC3 relationship, all of which may play a significant role in the connection behaviour. As the effect of the web in shear is not taken into account by Frye and Morris, no difference is envisaged between connections to internal and external columns. Furthermore, the bolts in tension are believed to affect the joint stiffness and limit the rotational capacity, and therefore are necessary in the formula.

It may not be feasible to include all parameters into a prediction equation for use in a design office or in an analysis program. For instance, the effect of bolt preloading or lack of fit cannot yet be taken into account in such approaches. However, it is essential to include at least the more influential connection components in the prediction equations. The Frye & Morris formulae have the advantage of being easy to use due to the limited dimensional parameters. It is also easy to be programmed in any computer analysis.

Fig. 3-7 shows the moment-rotation characteristics of ten typical joints in the 6storey,2bay frame using the Frye & Morris equation for extended end-plate connections without column stiffeners. The curves are nondimensional; the resistances of connections have been non-dimensionalized with respect to two-thirds of the beam's plastic moment resistance. The rotations are divided by the amount of rotation at two-thirds of the beam's plastic moment resistance. In some of the curves, the stiffness is seen to have increased in the range of medium moment. Experimental results show consistantly deteriorating stiffness due to increase in moment. Ang & Morris(1984) have also found that the equations sometimes give an increasing stiffness. They have suggested an exponential function to replace the polynomial. It then has the advantage of always yielding a slope which corresponds to the observed behaviour.

The EC3 prediction equation for the joint stiffness has been discussed by Moore(1989). He made comparison between experimental results and the moment-rotation curves produced by EC3 and concluded that this method gives a safe design moment but overestimates the flexibility of those joints which fail in a ductile manner. He has then suggested that the calculated $M-\phi$ curve should only be used to predict the beam and column displacements at serviceability.

A number of prediction equations, including that of EC3, assume a plateau for the moment-rotation curve of the connection. The Frye & Morris equation results in a curve with decreasing slope at the higher moments but without a plateau. This is similar to the experimental results where ductile failure occurs. The moment-rotation curves of three typical connections of the 6storey,2bay frame are plotted in Fig. 3-8, to compare the Frye & Morris and EC3 methods. It is seen that the initial stiffness of the former is less than the latter, while the ultimate moment of the latter is less than the former.

As described in 3-2-3-2, the sways resulted from EC3 are more than those from Frye & Morris formula. To explain the cause, reference should be made to Fig. 3-9. The connection moments resulted from the elastic-plastic analysis using EC3 have reached the ultimate resistance moment. Therefore, the stiffnesses used in the analysis correspond to points along the plateau. On the other hand, the moments of the similar joints represented by the Frye & Morris equation are much higher, and therefore correspond to a secant stiffness which is greater than that of EC3. The greater stiffnesses then result in less sway.

3-2-5-A Criterion for Serviceability Limit State

The above analyses show that sway deflections in semi-rigid frames designed by the wind-moment method are significantly larger than those calculated assuming rigid joints. The significance of sway deflections of unbraced frames has been found to be strongly influenced by the ratio of gravity load to wind load (Moy(1974) and Reading(1989)). Even though the design of some frames may be governed by strength, for others serviceability governs the design. The frames designed in the past by the wind-moment method according to existing serviceability limits do not exhibit distress in practice. Based on these statements, a criterion for frames designed with the wind-moment method can be obtained.

As shown in Table 3-6, the increase in sway was dependent on the moment-rotation relationship being used, and the deflection would be greater if the $M-\phi$ relationship was presented by the multilinear approximation of EC3 compared to the polynomial presentation of Frye and Morris. Based on discussions made in 3-2-4, the Frye & Morris formula for stiffened extended end plate connection was used in the following analyses, as it conforms more closely to the experimental behaviour of connections.

Two frames (4storey,2bay and 4storey,4bay) were designed by wind-moment method subjected to the following loading:

Roof: Dead load = 3.75 kN/m^2

Imposed load = 1.50 kN/m^2

Floors: Dead load = 4.50 kN/m^2

Imposed load = 6.00 kN/m^2

The frames were assumed to be at 6.0 m. centres longitudinally. Wind speed was taken as 45m/s and the load combination according to BS5950:Pt 1 for SLS, i.e. Dead+0.8(Imposed+Wind). The frames were assumed rigid and first-order analysis was undertaken. Then a semi-rigid analysis using the Frye & Morris representation for extended end-plate connections was carried out for each frame. The configuration of the frames and the results obtained are shown in Fig. 3-10. An increase of 42% for the 4storey,2bay frame and an increase of 37% for the 4storey,4bay frame were obtained.

To investigate the effect of variation in the wind load, a separate study was carried out for the 4storey,2bay frame subjected to wind loads due to various wind speeds from 38 to 52m/s. For semi-rigid analysis, the same representation of joint behaviour was used, namely the Frye & Morris equation. The results of rigid and semi-rigid analyses are given in Table 3-7. It can be seen that the increase of sway is limited between 42 and 44%. This implies that the effect of connection flexibility on sway is much more significant than the wind loading.

From the above investigations and those reported by Reading(1989) and Kavianpour(1990) the following can be concluded for semi-rigid deflections at SLS. If the common limit of sway, i.e. 1/300 of the height, is currently accepted, then the calculated rigid deflections of frames designed by the wind-moment method can be increased by 50% to allow approximately for the effect of connection flexibility. On the other hand, when a rigidly designed frame is analysed as semi-rigid, the conventional limitation of sway can be relaxed. As far as is known, structures incorporating semi-rigid frames designed by the wind-moment method and analysed for sway as rigid structures, have performed satisfactorily in practice. An increase of sway in the range of 25 - 60% has been reported here and elsewhere (Ackroyd & Gerstle(1982) and Reading(1989)) for frames designed by the wind-moment method comprising extended end plate joints (which is considered as an upper limit to semi-rigid connections). This implies that the sway limit of 1/300 may be relaxed by up to 25%. Therefore the suggestion of 1/270 for sway limit may not be unconservative.

3-3-Calculation of Sways in Unbraced Frames by Approximate Methods

A number of approximate methods are available for the calculation of sway, some of which enable direct design to specific limits. If a given frame is to be analysed for sway, the charts produced by Wood & Roberts(1975) can be used. An alternative solution is due to Moy(1974) which has the advantage of providing guidance on the changes that will be required in section properties if the deflections in a trial design are excessive.

However, if control of sway is likely to govern the member sizes, then equations due to Anderson & Islam(1979a)(1979b) enable a suitable design to be obtained directly. In order to choose the most convenient procedure in design, it is helpful to know at any early stage whether ultimate strength or serviceability limit on sway will dominate the choice of sections. Guidance on this for rigid jointed frames has been given by Anderson & Lok(1983).

The following sub-sections consider the simplified methods of sway calculation for either prediction of the dominant limit state or a quick estimation of horizontal deflection.

3-3-1-Derivation of Rigid Analysis

The Wood & Roberts method has been included in ECCS Recommendations(1978). The analysis is based on the frame shown in Fig. 3-11 which acts as the substitution for an individual storey of a multistorey frame. Such an analysis can also be used to check deflections in the top or bottom storeys. The spring of stiffness S represents the cladding stiffness.

By writing equilibrium equations and using the slope-deflection equations in terms of θ_t , θ_b and Δ , an expression for sway can be derived in terms of θ_t and θ_b. Joint equilibrium determines the unknown rotations θ_t and θ_b in terms of stiffness of connected members, therefore a non-dimensional expression for sway, $\overline{\Phi}$, is obtained in terms of k_t

and k_b (see Fig. 3-12(a)):

$$\overline{\Phi} = \frac{\frac{\Delta}{h}}{\frac{Fh}{12EK_c}} = f(k_b, k_t) \tag{3.10}$$

in which

$$k_t = \frac{K_c}{K_c + K_{bt}} \qquad \text{and} \qquad k_b = \frac{K_c}{K_c + K_{bb}}$$

For the sake of convenience, Wood and Roberts presented their analysis in the form of charts such as Fig. 3-12(b). More details can be obtained from Anderson(1983).

3-3-1-1-Substitute Frame

To use the above analysis, each storey of the actual frame must be replaced by a substitute structure having the form of Fig. 3-12(a). This is done by first transforming the actual frame into a substitute beam-column structure. Fig. 3-13 shows the substitute structure for a 6storey,2bay frame used later in this section. Two assumptions have been made for the substitute frame as follows:

a) The rotations of all joints at any one level are approximately equal for horizontal loading on the real frame.

b) Each beam restrains a column at both ends.

Accordingly, for a typical beam in the real frame, the slope-deflection equations can be written by the assumption:

$$\theta_A = \theta_B = \theta$$

which gives the end moment as:

$$M = 2E\left(\frac{I_b}{L}\right)(2\theta_A + \theta_B) = 6E\left(\frac{I_b}{L}\right)\theta \tag{3.11}$$

In the substitute frame (Fig. 3-11):

$$M = 2E\left(\frac{I_s}{L}\right)(2\theta) = 4E\left(\frac{I_s}{L}\right)\theta \tag{3.12}$$

where I_s is the second moment of area of beam in the substitute frame. Comparing Eqn. (3.11) with (3.12), the total stiffness K_b of a beam in the substitute frame is:

$$K_b = \sum 2(\frac{1.5I_b}{L}) = \sum 3(\frac{I_b}{L}) \tag{3.13}$$

and for a column in substitute frame:

$$K_c = \sum(\frac{I_c}{h}) \tag{3.14}$$

In both cases the summation is over all the beams or columns in the real frame at the level being considered.

The continuity of columns in a multistorey structure is allowed for by the modification of the distribution coefficients. As each beam restrains the column lengths above and below its level:

$$k_t = \frac{K_c+K_u}{K_c+K_u+K_{bt}} \quad \text{and} \quad k_b = \frac{K_c+K_l}{K_c+K_l+K_{bb}} \tag{3.15}$$

where K_u and K_l are shown in fig. 3-12(a) as the stiffness of upper and lower column in respect to the storey under consideration.

3-3-1-2-Examples

Three frames have been chosen as shown in Fig. 3-14 and designed by the wind connection method using Grade 43 steel. First the method explained in section 3-3-1-1 was employed to calculate the sway of each storey, then "exact" rigid analysis was carried out by means of an elastic analysis program (Benterkia(1991)). In the calculation of the sways, only unfactored wind loads were applied. The results are tabulated in Tables 3-8 to 3-10. In these tables the shear force at each storey, i.e. at the top of each column in the substitute frame, is given. The stiffnesses of column, top beam and bottom beam in the substitute frame are calculated according to Eqns. (3.13) to (3.15). By using these values in the chart of Fig. 3-12 for $\bar{s}=0$ (cladding ignored), $\bar{\Phi}$ and consequently Δ is obtained from Eqn. (3.10) as:

$$\Delta = \frac{Fh^2}{12EK_c}\bar{\Phi} \tag{3.16}$$

Finally, the results of "exact" analysis are given in the last column of the tables.

Comparison can be made between these results for each individual storey. It is shown that the results obtained by Wood's method are higher than those obtained from "exact" analysis for low rise frames, while they are lower for the medium rise frames. The overall sway can be compared, resulting in less than 20% difference for the 3storey,1 and 2 bay frames and about 2% for the 6storey,2bay frame (Table 3-11). As the inaccuracy of this method is about 20% overestimation and 2% underestimation of sway deflection for the above frames, it may be considered as a sufficiently safe way for a rapid prediction of sway in the absence of sophisticated computer programs.

3-3-2-Derivation of Semi-Rigid Analysis

Where the practical frames are concerned, account must be taken of the flexibility of connections which affect the effective stiffness of the beams. A simplified case was presented by Driscoll(1976), in which the "absolute stiffness" of a member AB for joint rotation at A is given as follows (see Fig. 3-15):

a) For "rigid frame analysis", i.e. stiffness of a beam with the far end B fixed, as used in moment distribution, $\theta_{AB}=1$ and $\theta_{BA}=0$:

$$K_{AB} = M_{AB} = 4E\left[\frac{3(2K_B+1)}{4(K_A+1)(K_B+1)-1}\right]\frac{I}{L} \tag{3.17}$$

b) For use in a frame free to sidesway, $\theta_{AB}=1$ and $\theta_{BA}=\theta_{AB}$:

$$K_{AB} = M_{AB} = 6E\left[\frac{2(K_B+1)+1}{4(K_A+1)(K_B+1)-1}\right]\frac{I}{L} \tag{3.18}$$

c) For use in a frame prevented from sidesway , $\theta_{AB}=1$ and $\theta_{BA}=-\theta_{AB}$:

$$K_{AB} = M_{AB} = 6E\left[\frac{2(K_B+1)-1}{4(K_A+1)(K_B+1)-1}\right]\frac{I}{L} \tag{3.19}$$

where $K_A = 3Z_A(\frac{EI}{L})$

$K_B = 3Z_B(\frac{EI}{L})$

$Z = \frac{\phi}{M}$

In the above formulae, Z can be replaced by the connection stiffness C, as one is the inverse of the other.

3-3-2-1-Special Case in a Sway Frame

If the rotation at joints at each storey level is assumed equal ($K_A = K_B$), as is assumed in deriving the substitute frame of Fig. 3-12(a), the corresponding equation for the "relative stiffness" can be obtained by dividing the "absolute stiffness" by E and an appropriate constant. For unbraced frames from Eqn. (3.18):

$$K_r = \left[\frac{1}{2K_B+1}\right]\frac{I}{L} \tag{3.20}$$

and if the connection stiffness is used:

$$K_r = \left[\frac{1}{\frac{6EI}{CL}+1}\right]\frac{I}{L} \tag{3.21}$$

K_r is therefore the beam stiffness to be used in analysis to account for the semi-rigid end connections.

The result of using Eqn. (3.20) or (3.21) is generally accurate for one bay frames as reported by Driscoll. It will be shown in the following examples that the above assumption is not far from accuracy even for multibay frames as long as the spans are equal. For determination of the "relative stiffness" of the beam restrained by semi-rigid connections, it is sufficient to multiply the beam stiffness by the relative stiffness ratio. A rigid frame with beams of "relative stiffness" can then be analysed to find the sway displacements.

3-3-2-2-Examples

The same frames shown in Fig. 3-14 were used. They were designed by the wind-moment method with stiffened extended end plate connections. The frames were analysed by an "exact" elastic semi-rigid program (Benterkia(1991)) with the connection stiffness C taken as the initial stiffness of joint found by EC3 method. The frames were also analysed rigidly with the beams' stiffness taken as the "relative stiffness". The results will be described separately for each frame.

a) Three storey, one bay frame:

For this frame K_r was calculated as 0.58 of the beam stiffness. The second moment of area of the beam was multiplied by this factor and used in rigid analysis to find the storey sways. The maximum horizontal displacement of 2.978 cm. was found, which is very close to semi-rigid deflection (2.976 cm.) of the real frame.

b) Three storey, two bay frame:

This frame was treated differently, as in a precise approach the rotation of beam ends cannot be assumed equal for the internal and external joints. Therefore K_{AB} and K_{BA} were calculated in which unequal Z_A and Z_B were used. Nevertheless, the two values of "relative stiffness" obtained were quite close (0.59 and 0.61 of the beam stiffness) which implies the possibility of taking an average of K_{AB} and K_{BA} as the "equivalent beam stiffness". The results are sufficiently close to those of real frame. An overall drift of 1.701 cm. was found in comparison with 1.705 cm. from exact analysis.

c) Six storey, two bay frame :

The same procedure as that described in (b) was carried out for this frame. The consistency of results was interesting. A total sway deflection of 7.862 cm. was found, in comparison with 7.857 cm. from exact analysis.

3-3-3-Equivalent Stiffness Used in Substitute Frame

The author suggests that as an alternative, the beam stiffnesses calculated by Driscoll's method can be used in Wood's substitute frame to find the horizontal deflections at each storey. This has been examined on the 6storey,2bay frame. The calculations are set out in Tables 3-12 and 3-13.

The maximum sway at roof level is obtained by the summation of storey sways. The resulting deflection of 7.80 cm. compares with the "exact" value of 7.86 cm. This again justifies the accuracy of the methods described and confirms that the methods of Driscoll and Wood can be combined as a "desk method" for estimation of sway in semi-rigid frames.

3-4-Semi-Rigid Action at Collapse

In the previous sections, different aspects of semi-rigid effects on frames at serviceability were considered. The internal elastic moments were found to be highly influenced by the semi-rigid joint behaviour. A separate study was undertaken on the ultimate response of semi-continuous frames, to consider the effects of partial strength connections on the frame stability.

Frames with flexible connections reach the permissible deformation or allowable stress at a lower load than frames with rigid connections. Due to the relative displacements of the columns, extra moments are introduced in the frame as shown in Fig. 3-16. The $P-\Delta$ effect is not a specific property of frames with semi-rigid connections, but it should be taken into account when a frame is considered at the ultimate limit state.

Fig. 3-17(a) shows the behaviour of an unbraced frame with rigid and full-strength connections, while Fig. 3-17(b) depicts the behaviour of the same frame but with semi-rigid (and partial strength) connections. The real behaviour of such frames is shown with the thick lines in Fig. 3-17. As shown, the elastic critical load of the frame decreases as the flexibility of connections increases. The bearing capacity of the frame, based on first-order plastic theory decreases as the strength of connections decreases. The maximum loading capacity of frame will decrease as the flexibility and resistance of connections decrease. phenomena.

In the plastic analysis of semi-rigid steel frames, the plastic hinges may form at the connections, beams and columns. The sequence of the formation of plastic hinges depends on the performance of the beams and columns, as well as the semi-rigid connections. A plastic hinge forms at a connection when the plastic resistance moment of the connection has been attained. Therefore, the plastic response of the frame is dependent on the strength of the joints.

In this study, the collapse load levels of a three storey, one bay frame were calculated by means of a second-order elastic-plastic analysis program (Kavianpour(1990)) taking into account the connection flexibility. The investigation was carried out for "full strength" and "partial strength" connections with respect to the moment resistance of the connected beam.

3-4-1-Full Strength Connection

The frame shown in Fig. 3-18 was analysed with variable joint stiffness while the applied loads were constant. The frame is similar to that used by Jaspart(1988) but the loading is as shown in Fig. 3-18. All joints within the frame were assumed to have the same secant stiffness. Second-order elastic-plastic analysis was carried out with various connection flexibilities to find the load level at collapse. For the sake of simplicity, the reduction in the plastic resistance moment of the columns due to axial force was neglected. The load level at collapse is plotted against the joint stiffness in Fig. 3-19.

As shown, the semi-rigid joint action at collapse is not as influential as its effect on serviceability as described earlier. This is due to the redistribution of moments resulted from plasticity, which occurs when the ultimate capacity of frame is achieved. This has also been found in the research done by Chikho & Kirby(1989). They analysed a two storey,one bay frame (Fig. 3-20(a)) at incremental load levels. For both rigid and semi-rigid analysis, almost the same amount of moment was developed at the windward end of each beam as the collapse was reached. This is shown in Fig. 3-20(b). They reported negligible reduction in the collapse load level for low-rise frames. Whilst for the more slender three storey frame, Jaspart(1988) reported 18% reduction. For a single storey frame with welded connections analysed by Tschemmernegg & Humer(1988), a reduction of only 4% was found.

The main influence of semi-rigid but full strength joints is therefore restricted to an increase in second-order effects. In consequence, reductions in the load level at collapse, compared to rigid-jointed frames, are less dramatic than increases of sway at service load. Although a full strength joint reduces the need for the connection ductility, the additional cost for providing such a strong connection may not be justified.

3-4-2-Partial Strength Connection

The same frame as shown in Fig. 3-18 was examined with identical joints at the beam's ends. To represent a nominally rigid but partial strength joint, the bilinear representation of $M-\phi$ characteristic was assumed as shown in Fig. 3-21, based on the limiting stiffness of nominally rigid joints as defined by EC3. The resistance of the connections was assumed as a varying fraction of the beam resistance.

The analysis was initiated with full strength connections ($\bar{m}=1.0$). Gradual decreases were then made in the connection resistance and the relevant failure load factor was determined at each step. The analysis program (Kavianpour(1990)) was based on second-order plastic hinge theory. The collapse load level is plotted against the connection resistance in Fig. 3-22.

From comparison between Figs. 3-19 and 3-22 it follows that the influence of partial strength joints on the collapse load is more significant than semi-rigid but full strength joints. This fact has also been concluded by Zandonini(1986) from investigations on portal frames. This becomes more obvious if the stiffness of an extended end-plate connection (ECCS(1992)) given in Fig. 3-23 is indicated in Fig. 3-19.

The results from a single storey frame with flush end plate connections are also reported by Chikho & Kirby(1989). A 14% reduction in the collapse load level was observed compared to rigid analysis. Reading(1989) reported a reduction of 20-35% for a series of multistorey frames with flush or extended end plate unstiffened connections.

With partial strength connections, a high degree of moment redistribution may occur, because of the limited resistance of the connections. Thus, high rotation capacity may be required to develop the predicted performance.

An example of the mode of collapse, for the particular case of the joint resistance limited to 0.8 of the plastic moment capacity of the beam, is shown in Fig. 3-24.

3-5-Conclusions

From the study on semi-continuous frames described in this chapter, the following conclusions can be deduced:

1) The EC3 definition for rigid joints in unbraced frames exceeds the rigidity of stiffened extended end-plate connection, which in practice is recognized as a stiff connection. As fully-welded and T-stubs connections are not in common use, it is concluded that all steel connections in practice must be treated as semi-rigid or flexible joints according to EC3.

2) The EC3 prediction equation for connection stiffness underestimates the moment resistance of extended end plate joints, and in general results in more flexibility for end plate connections. These will cause greater horizontal deflections and lower collapse loads for frames compared to practical behaviour of joints.

3) Regarding the EC3 definition for rigid connections, the increase of about 10% in the horizontal deflection due to semi-rigid analysis suggests of a modified allowable sway ratio of 1/270 for unbraced medium-rise frames.

4) The Frye & Morris(1975) formula for extended end-plate connections does not give a precise indication of the connection behaviour because only a few parameters are taken into account. It ignores the effect of parameters such as the column web in shear, tension and compression zones. In comparison, EC3 has included the above factors as well as the bolts in tension. The initial stiffness of joints given by Frye & Morris formula for unstiffened extended end plate connections is less than that of EC3. The Frye & Morris equation may give increasing stiffness, while it is known from the test data that the connection stiffness deteriorates as the moment increases.

5) At the serviceability limit state, where the wind-moment method is used in design, rigid deflections obtained from elastic analysis can be increased by 50% to allow for the connection flexibility at serviceability limit state. The above correction is safely acceptable for all wind speeds.

6) The Wood & Roberts(1975) approximate method can be used as a simplified way to predict sway displacements. High-rise frames may need to be checked at serviceability by an exact method after the design is completed.

7) The method proposed by Driscoll(1976) to take account of connection flexibility by inclusion of the initial stiffness of the joints in the beam stiffness appears quite accurate in the sway calculation. The term "relative stiffness" can be used for a beam whose second moment of area is decreased to allow for the flexibility of joints at its ends.

8) The stiffness of a beam in a substitute frame can be replaced by the "equivalent stiffness" to enable the Wood & Roberts method to be used for semi-rigid frames.

9) The influence of the stiffness of semi-rigid joints on internal moments at collapse is less significant than its effect on sway at serviceability. Redistribution of moments occurs as the plasticity develops.

10) Partial strength joints have a more significant influence on the collapse load than semi-rigid but full strength joints. The limited resistance of the connections form localized points of weakness in the structure.

Table 3-1, Increase in sway assuming moment-rotation characteristics at the EC3 boundary line comparing rigid with semi-rigid analysis

FRAME	WIDTH OF BAYS	HEIGHT OF STOREYS (top to bottom)	INCREASE
8storey,1bay	1@9.00	7@3.50+1@4.50	11.3%
8storey,1bay	1@6.00	7@3.50+1@4.50	10.5%
8storey,1bay	1@4.50	7@3.50+1@4.50	10.3%
6storey,1bay	1@9.00	5@3.50+1@4.50	9.4%
6storey,2bay	2@9.00	5@3.50+1@4.50	8.0%
4storey,2bay	2@6.00	3@4.00+1@5.00	7.3%
2storey,4bay	4@9.00	1@5.00+1@6.00	2.7%

Table 3-2, The effect of reduction of the stiffness of beams on sway increase

FRAME	ORIGINAL BEAM SECTIONS	DECREASED BEAM SECTIONS	INCREASE OF SWAY	PREVIOUS INCREASE
8storey,1bay span=6.00 m. Grade 50	406x140UB39(roof) 457x191UB67(floors)	356x127UB33 457x152UB52	11.4%	10.5%
6storey,1bay span=9.00 m. Grade 50	457x152UB60(roof) 533x210UB92(floors)	457x152UB52 533x210UB82	12.2%	9.4%
6storey,1bay span=9.00 m. Grade 50	457x152UB60(roof) 533x210UB92(floors)	457x152UB52 457x191UB74	13.5%	11.2%
2storey,4bay spans=9.0 m. Grade 43	457x191UB67(roof) 610x229UB113(floors)	406x140UB46 533x210UB92	2.9%	1.8%
2storey,4bay spans=9.0 m. Grade 43	457x191UB67(roof) 610x229UB113(floors)	406x140UB46 457x191UB74	3.9%	1.8%

Table 3-3, The effect of changing column sections on sway differences due to semi-rigid analysis for an 8storey,1bay frame

NO. OF STOREY	CONTINUOUS SECTIONS	EXACT SECTIONS
1	305x305 UC 97	305x305 UC 97
2	305x305 UC 97	254x254 UC 89
3	254x254 UC 73	254x254 UC 73
4	254x254 UC 73	254x254 UC 73
5	203x203 UC 71	203x203 UC 71
6	203x203 UC 71	203x203 UC 60
7	203x203 UC 46	203x203 UC 46
8	203x203 UC 46	152x152 UC 30
INCREASE IN SWAY	10.5%	9.3%

Table 3-4, The effect of grade of steel on sway due to semi-rigid analysis

FRAME bay length storey height	GRADE OF STEEL	INCREASE IN SWAY
8storey,1bay 1@6.00 m. 7@4.50+1@6.00 m.	50	9.3%
	55	10.4%
2storey,4bay 4@9.00 m. 1@5.00+1@6.00 m.	43	1.8%
	50	2.7%

Table 3-5, Values of variables in Frye & Morris relationship (mm.)

FRAME	STOREY	BEAM DEPTH D*	END-PLATE THICKNESS t	COLUMN FLANGE THICKNESS f
8storey,2bay	roof	364.0	12.5	11.0
	7	528.3	15.0	11.0
	6 and 5	528.3	20.0	17.3
		528.3	20.0	15.4
	4	528.3	25.0	17.3
		528.3	25.0	15.4
	3 and 2	528.3	25.0	15.4
		528.3	25.0	23.8
	1	528.3	30.0	15.4
		528.3	30.0	23.8
6storey,2bay	roof	364.0	12.5	11.0
		364.0	12.5	14.2
	5	528.3	15.0	11.0
		528.3	15.0	14.2
	4	528.3	15.0	11.0
		528.3	20.0	14.2
	3,2	528.3	20.0	11.0
		528.3	20.0	14.2
	1	528.3	25.0	12.5
		528.3	25.0	18.7
4storey,2bay	roof	364.0	12.5	11.0
	3	528.3	15.0	17.3
	2 and 1	528.3	20.0	17.3

* D = *beam depth* $\qquad$ $d = D + 50.0mm.$

Table 3-6, Comparison of drifts resulted from EC3 and Frye & Morris equations with rigid sways

FRAME	RIGID SWAY (cm.)	SEMI-RIGID				EC3 COMPARED TO FRYE & MORRIS
		FRYE & MORRIS		EC3		
		SWAY (cm.)	RATIO TO RIGID	SWAY (cm.)	RATIO TO RIGID	
8storey,2bay	7.46	11.42	1.53	16.58	2.22	45%
6storey,2bay	5.47	8.16	1.49	14.03	2.56	72%
4storey,2bay	2.88	3.74	1.30	7.24	2.51	122%

Table 3-7, Sways of 4storey,2bay frame subjected to various wind loads

WIND SPEED m/s	RIGID OR SEMI-RIGID	DEFLECTION AT EACH FLOOR				TOTAL SWAY RATIO	SWAY INCREASE %
		1st	2nd	3rd	4th		
38	R	1.02	1.70	2.52	2.95	H/576	44
	S-R	1.28	2.34	3.51	4.23	H/401	
41	R	1.19	1.98	2.94	3.44	H/494	43
	S-R	1.49	2.72	4.07	4.92	H/345	
43	R	1.31	2.17	3.23	3.80	H/447	42
	S-R	1.64	2.99	4.48	5.41	H/314	
45	R	1.43	2.38	3.54	4.16	H/409	42
	S-R	1.79	3.28	4.90	5.92	H/287	
47	R	1.56	2.61	3.86	4.54	H/374	42
	S-R	1.95	3.56	5.34	6.46	H/263	
50	R	1.77	2.96	4.37	5.14	H/331	42
	S-R	2.20	4.03	6.04	7.30	H/233	
52	R	1.91	3.20	4.73	5.56	H/306	42
	S-R	2.38	4.36	6.52	7.90	H/215	

Table 3-8, Sway deflections for 3storey,1bay frame using approximate method

STOREY	F(kN)	$K_c(cm.^3)$	$K_b(cm.^3)$	$k_{bt}(cm.^3)$	$k_{bb}(cm.^3)$	k_t	k_b	$\overline{\Phi}$	$\Delta(cm.)$	$\Delta_{ex}(cm.)$
3	17.73	44	107	107	107	0.29	0.45	1.90	0.38	0.36
2	34.52	44	107	107	107	0.45	0.42	2.10	0.82	0.70
1	50.28	34	107	107	∞	0.39	0	1.42	1.73	1.46

Table 3-9, Sway deflections for 3storey,2bay frame using approximate method

STOREY	F (kN)	$K_c(cm.^3)$	$K_b(cm.^3)$	$k_{bt}(cm.^3)$	$k_{bb}(cm.^3)$	k_t	k_b	$\overline{\Phi}$	$\Delta(cm.)$	$\Delta_{ex}(cm.)$
3	18.67	76	214	214	214	0.26	0.42	1.78	0.22	0.20
2	36.34	76	214	214	214	0.42	0.39	2.00	0.48	0.40
1	52.93	59	214	214	∞	0.39	0	1.42	1.04	0.86

Table 3-10, Sway deflections for 6storey,2bay frame using approximate method

STOREY	F (kN)	$K_c (cm.^3)$	$K_b (cm.^3)$	$k_{bt} (cm.^3)$	$k_{bb} (cm.^3)$	k_t	k_b	$\overline{\Phi}$	$\Delta (cm.)$	$\Delta_{ex} (cm.)$
6	26.	39	317	317	317	0.11	0.20	1.28	0.42	0.50
5	54.	39	317	317	317	0.20	0.20	1.38	0.95	0.99
4	81.	67	317	317	317	0.30	0.30	1.65	0.99	1.00
3	106.	67	317	317	317	0.30	0.30	1.65	1.30	1.30
2	128.	144	317	317	317	0.48	0.45	2.30	1.02	1.08
1	149.	112	317	317	∞	0.41	0	1.45	1.59	1.54

Table 3-11, Comparison between overall sways

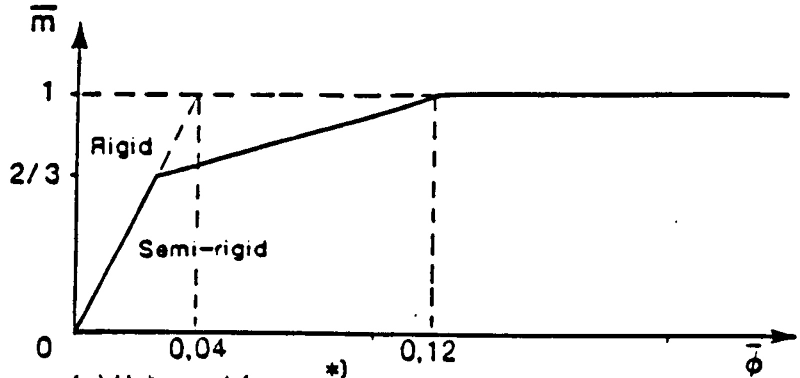
FRAME	APPROX. METHOD (cm.)	EXACT ANALYSIS (cm.)	DIFFERENCE
3storey,1bay	2.93	2.51	17% overestimation
3storey,2bay	1.75	1.47	19% overestimation
6storey,2bay	6.27	6.40	2% underestimation

Table 3-12, Calculation of equivalent beam stiffnesses for the 6storey,2bay frame

STOREY	Z_A	Z_B	K_A	K_B	$\bar{K}_{AB}$	$\bar{K}_{BA}$	$\bar{K}_r$	$I_{eq}\,(cm.^4)$
6 5	$\frac{0.00150}{6986}$	$\frac{0.00150}{6986}$	0.70	0.70	0.42	0.42	0.42	19950
4 3	$\frac{0.00148}{12469}$	$\frac{0.00120}{12469}$	0.38	0.31	0.58	0.60	0.59	28025
2 1	$\frac{0.00120}{12469}$	$\frac{0.00071}{13109}$	0.31	0.18	0.65	0.70	0.675	32062

Table 3-13, Sway deflections for semi-rigid 6storey,2bay frame using approximate method

STOREY	F (kN)	$K_c (cm.^3)$	$K_b (cm.^3)$	$k_{bt} (cm.^3)$	$k_{bb} (cm.^3)$	k_t	k_b	$\overline{\Phi}$	$\Delta (cm.)$	$\Delta_{ex} (cm.)$
6	26.	39	133	133	133	0.23	0.37	1.64	0.54	0.68
5	54.	39	133	133	133	0.37	0.37	1.90	1.31	1.25
4	81.	67	187	187	187	0.42	0.42	2.03	1.22	1.26
3	106.	67	187	187	187	0.42	0.42	2.03	1.60	1.59
2	128.	144	214	214	214	0.57	0.54	2.86	1.27	1.35
1	149.	112	214	214	∞	0.54	0	1.70	1.86	1.73

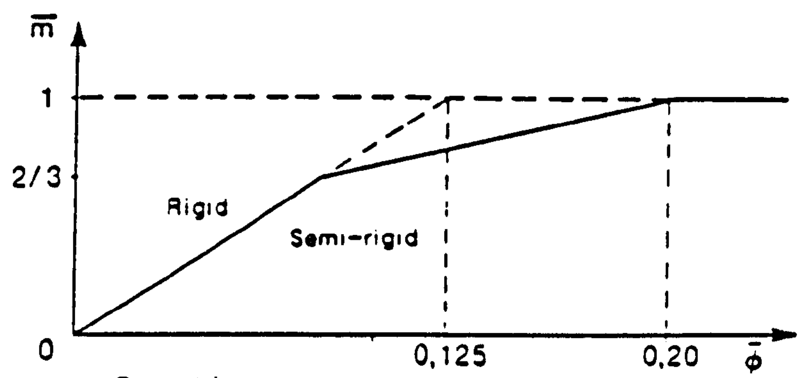


(a) Unbraced frames *)

when $\overline{m} \leq 2/3$: $\overline{m} = 25\,\overline{\phi}$

when $2/3 < \overline{m} \leq 1{,}0$: $\overline{m} = (25\,\overline{\phi} + 4)/7$

*) but see also 6.9.6.2(5)



(b) Braced frames

when $\overline{m} \leq 2/3$: $\overline{m} = 8\,\overline{\phi}$

when $2/3 < \overline{m} \leq 1{,}0$: $\overline{m} = (20\,\overline{\phi} + 3)/7$

$$\overline{m} = \frac{M}{M_{p\ell Rd}} \quad : \quad \overline{\phi} = \frac{E\,I_b\,\phi}{L_b M_{p\ell Rd}}$$

Fig. 3-1, Classification of connections to EC3

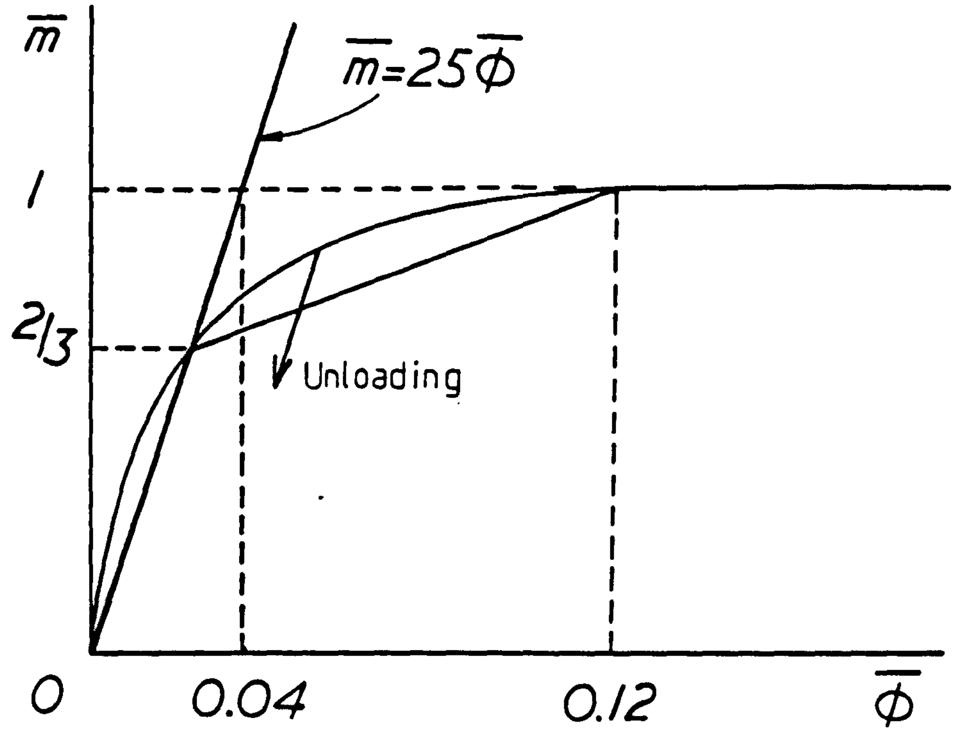


Fig. 3-2, The M–φ behaviour used in the study on EC3 classification of connections

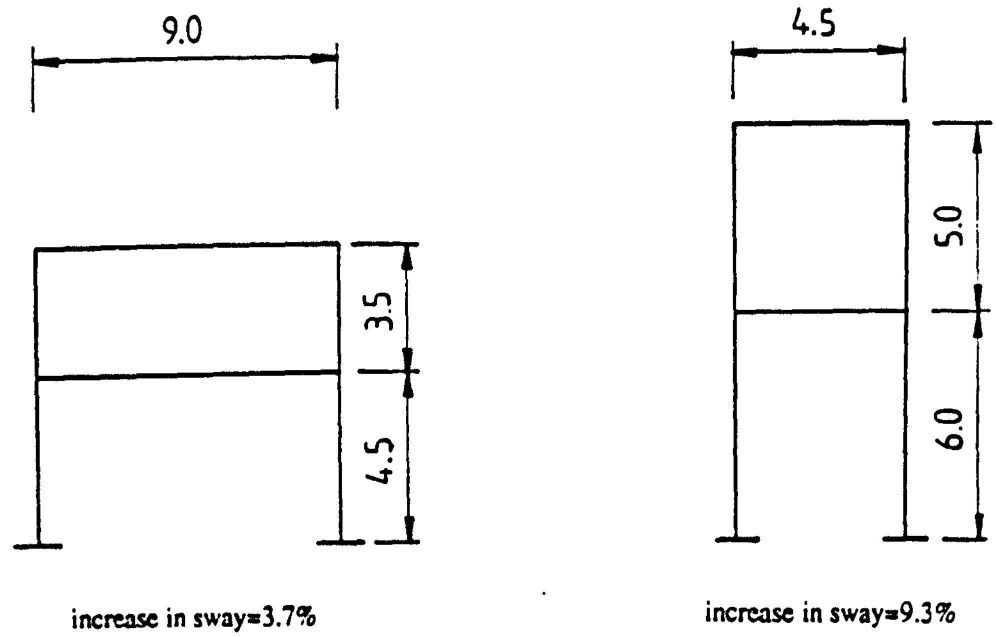


Fig. 3-3, Initial frames used in the study on EC3 classification of joints

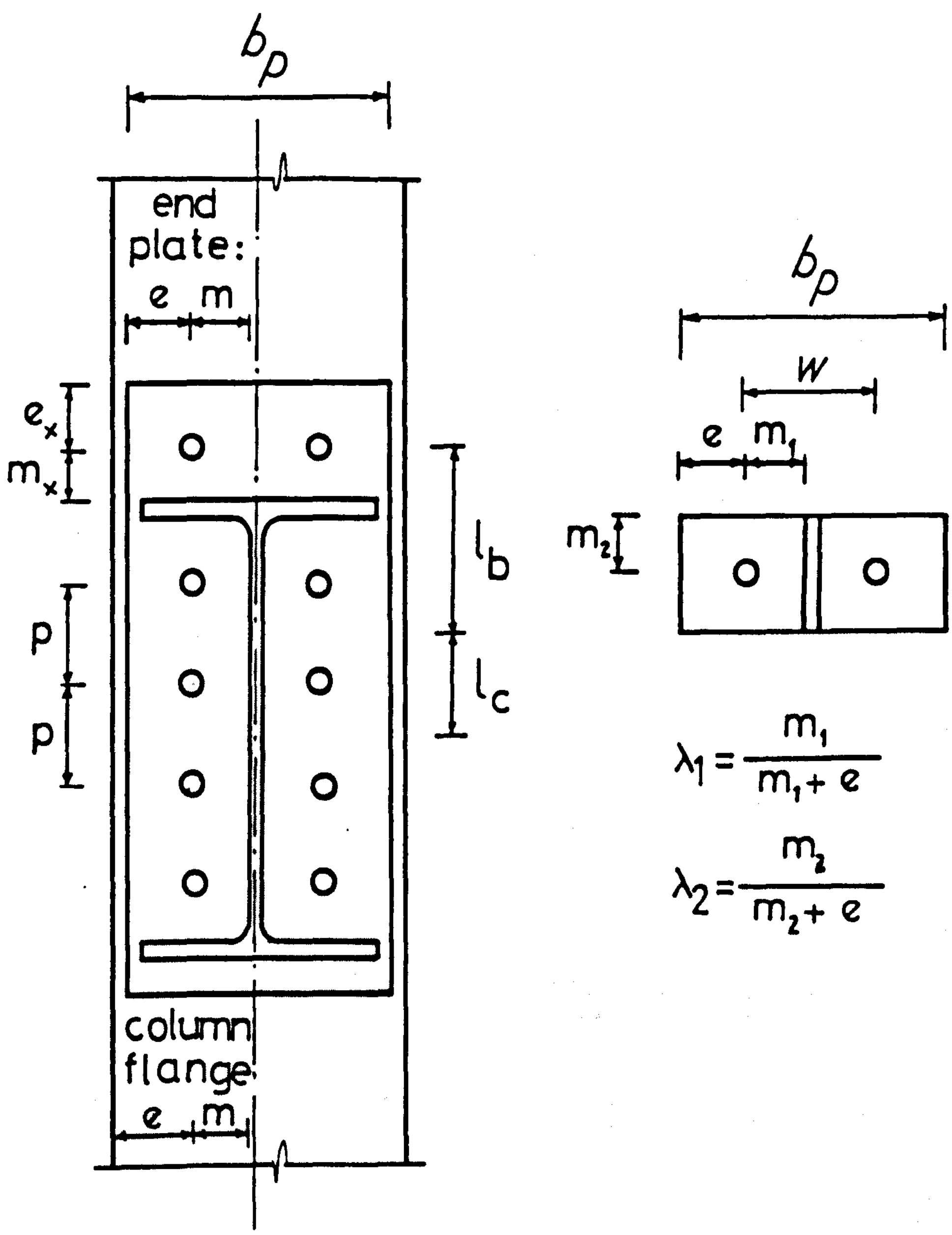


Fig. 3-4, Notations used in Annex J of EC3 for end plate connections

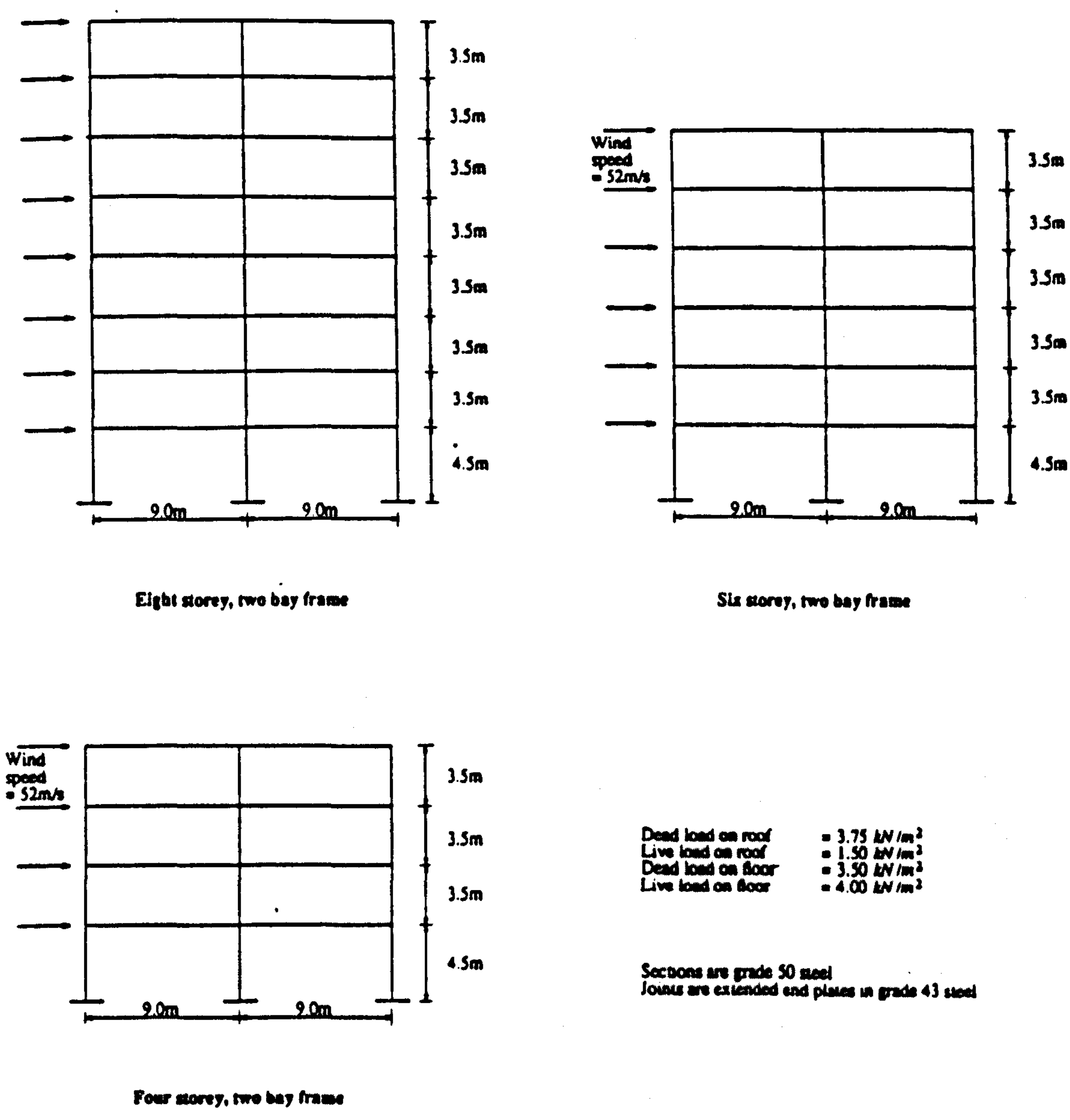


Fig. 3-5, Geometry and loading details of three basic frames

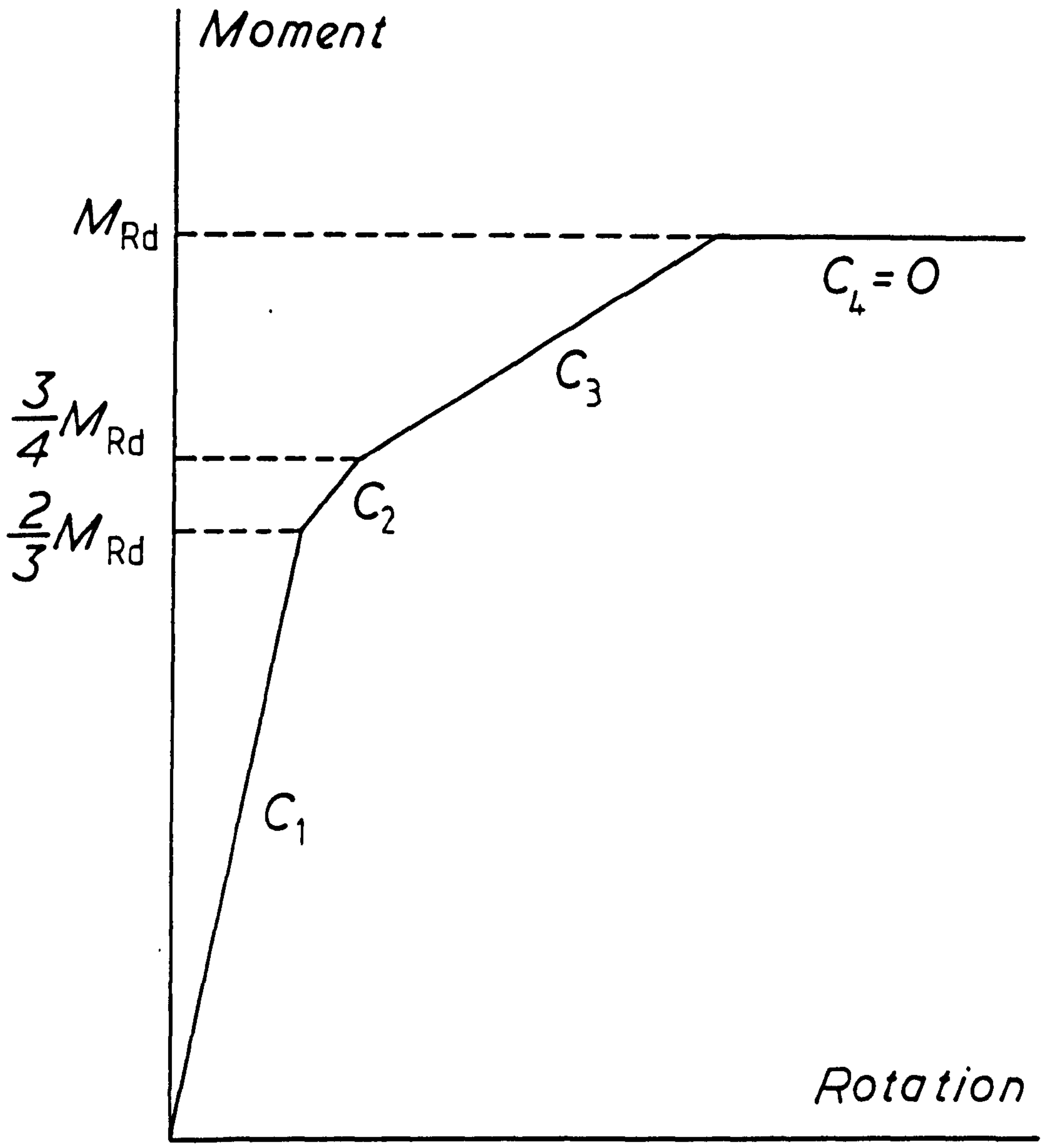


Fig. 3-6, The moment-rotation behaviour used in the sway analysis

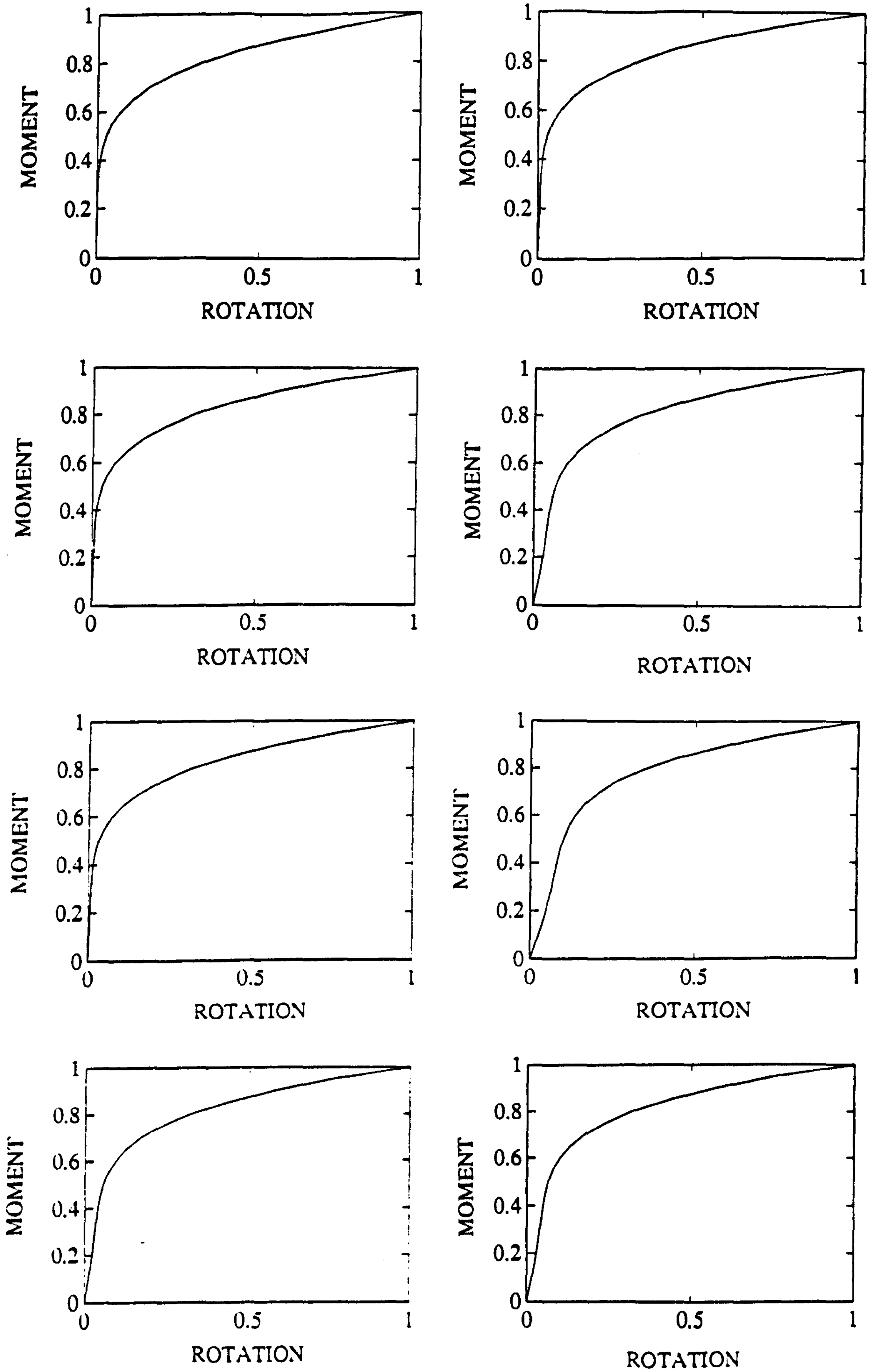


Fig. 3-7, The moment-rotation curves for eight connections of 6storey,2bay frame using Frye & Morris relationship for extended end plate joints

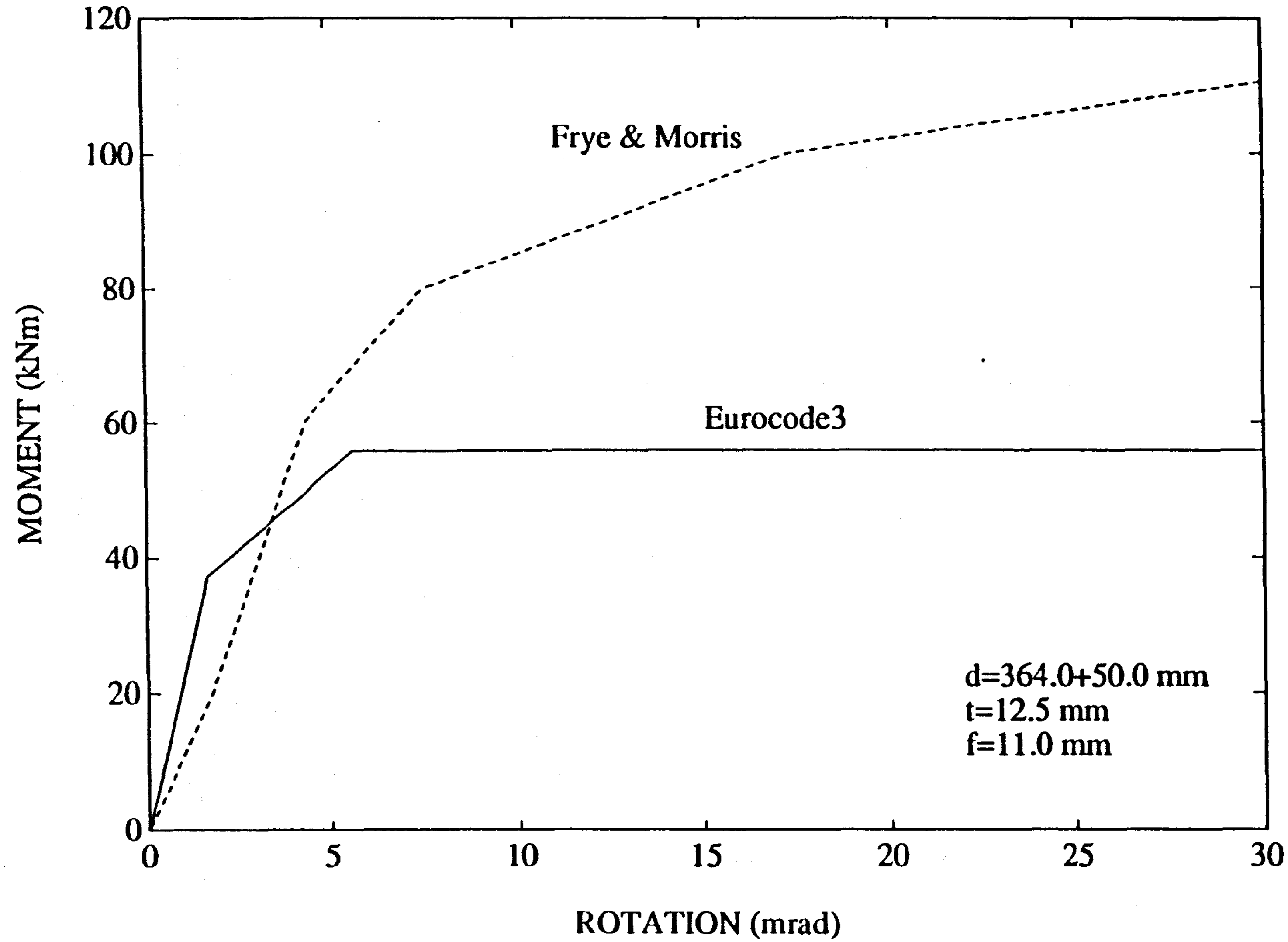


Fig. 3-8(a), comparison between moment-rotation curves obtained from Frye & Morrois and EC3 formulae

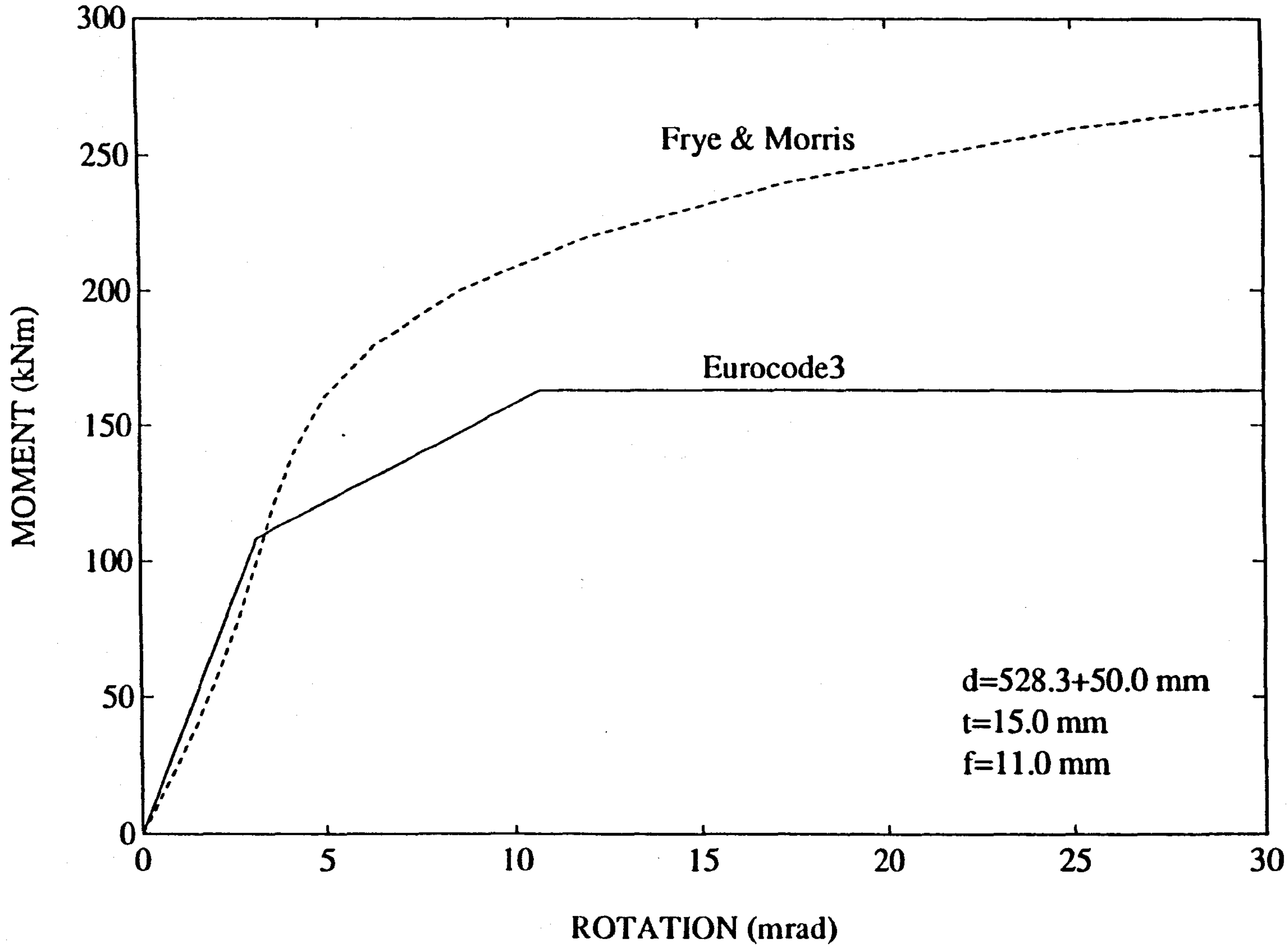


Fig. 3-8(b), comparison between moment-rotation curves obtained from Frye & Morrois and EC3 formulae

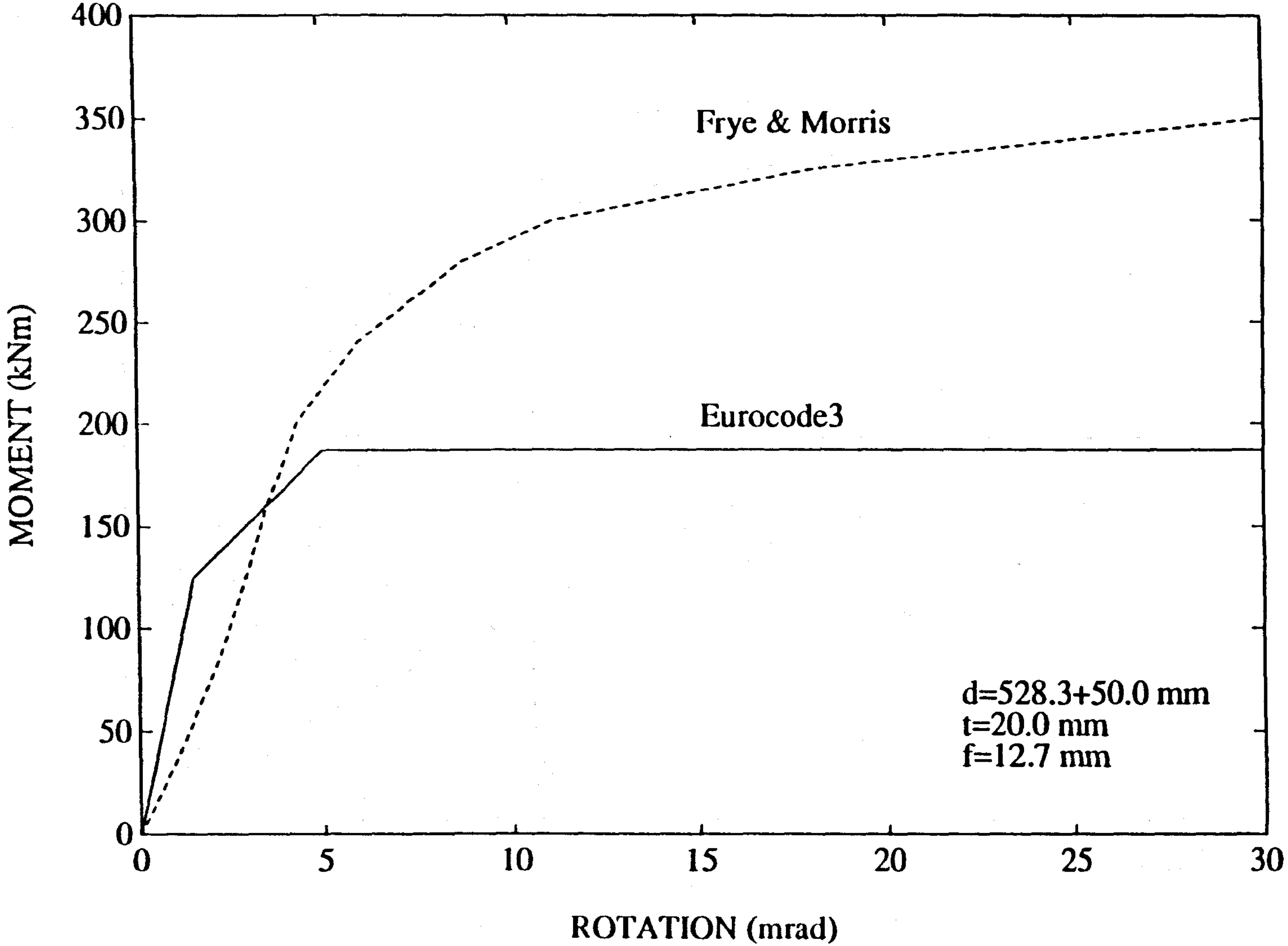


Fig. 3-8(c), comparison between moment-rotation curves obtained from Frye & Morrois and EC3 formulae

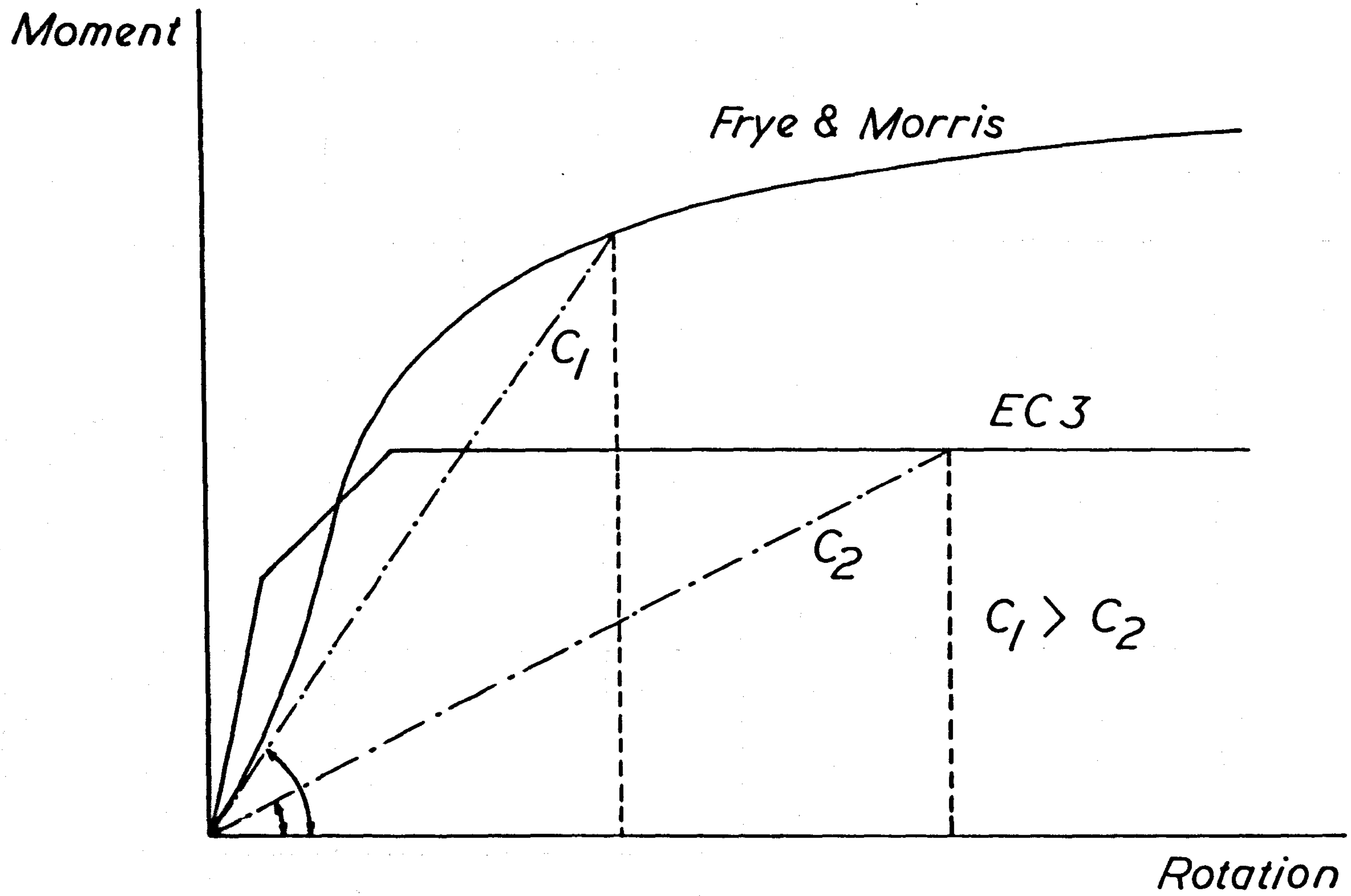


Fig. 3-9, Stiffness of connection in the plastic analysis of frame using Frye & Morrois and EC3 formulae

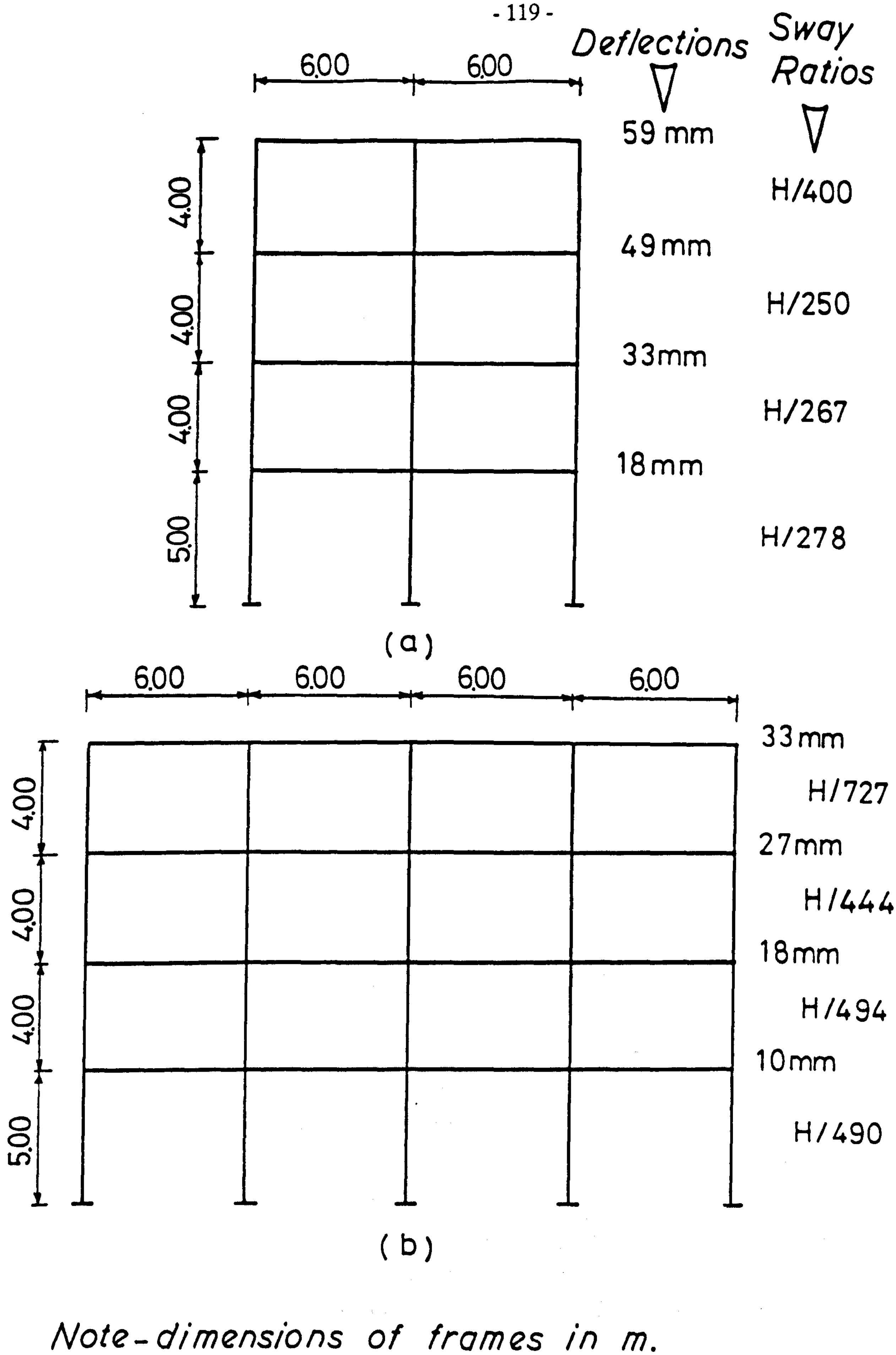


Fig. 3-10, Geometry, deflections and sway ratios of two basic frames

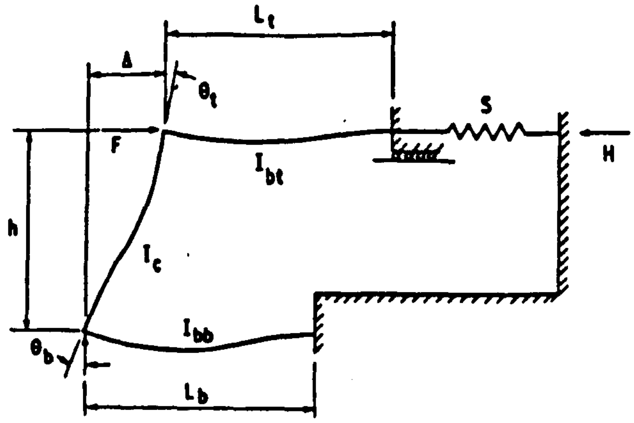


Fig. 3-11, Single storey substitute frame

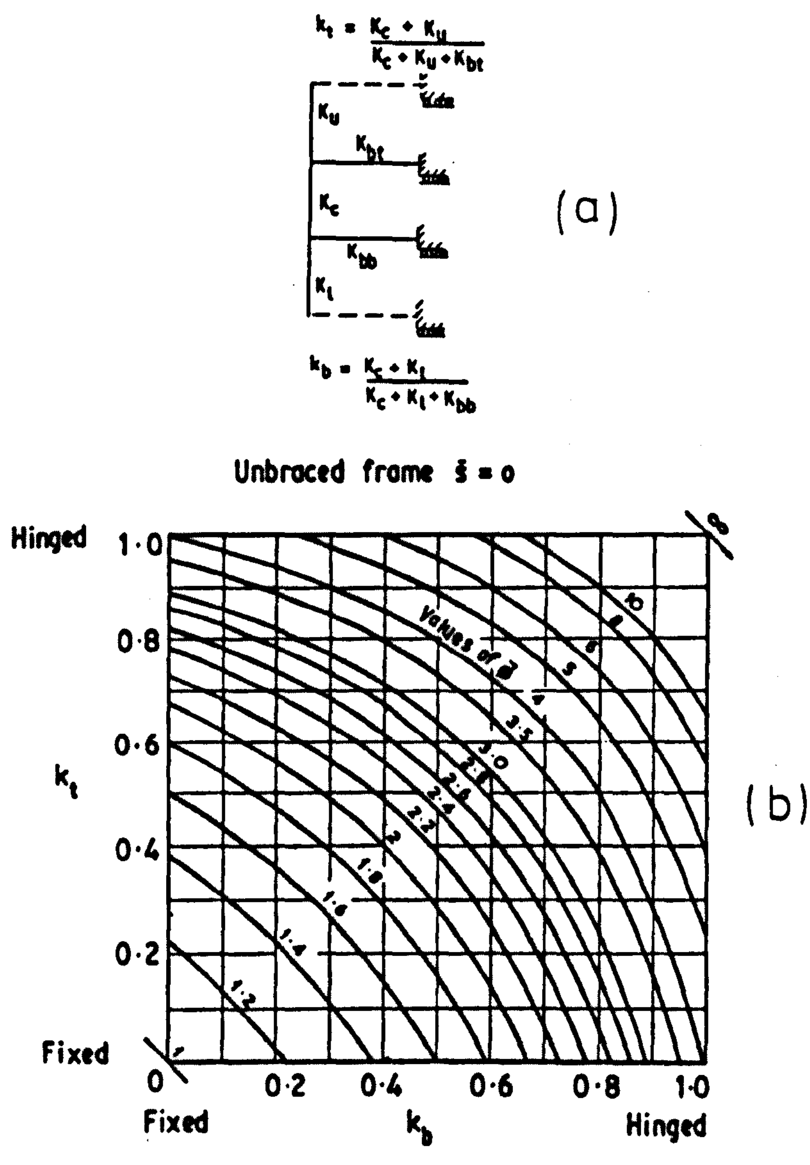


Fig. 3-12, Sidesway deflection for unbraced frame, (a)distribution coefficients, (b)values of $\bar{\Phi}$ (unbraced frame, $\bar{s}$=0)

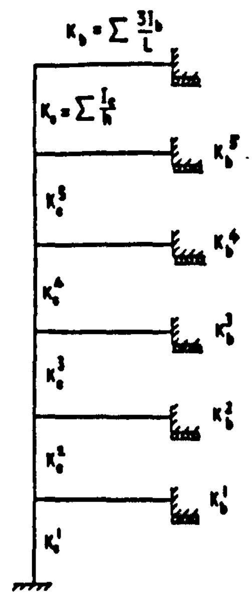


Fig. 3-13, Substitute frame for 6storey,2bay structure

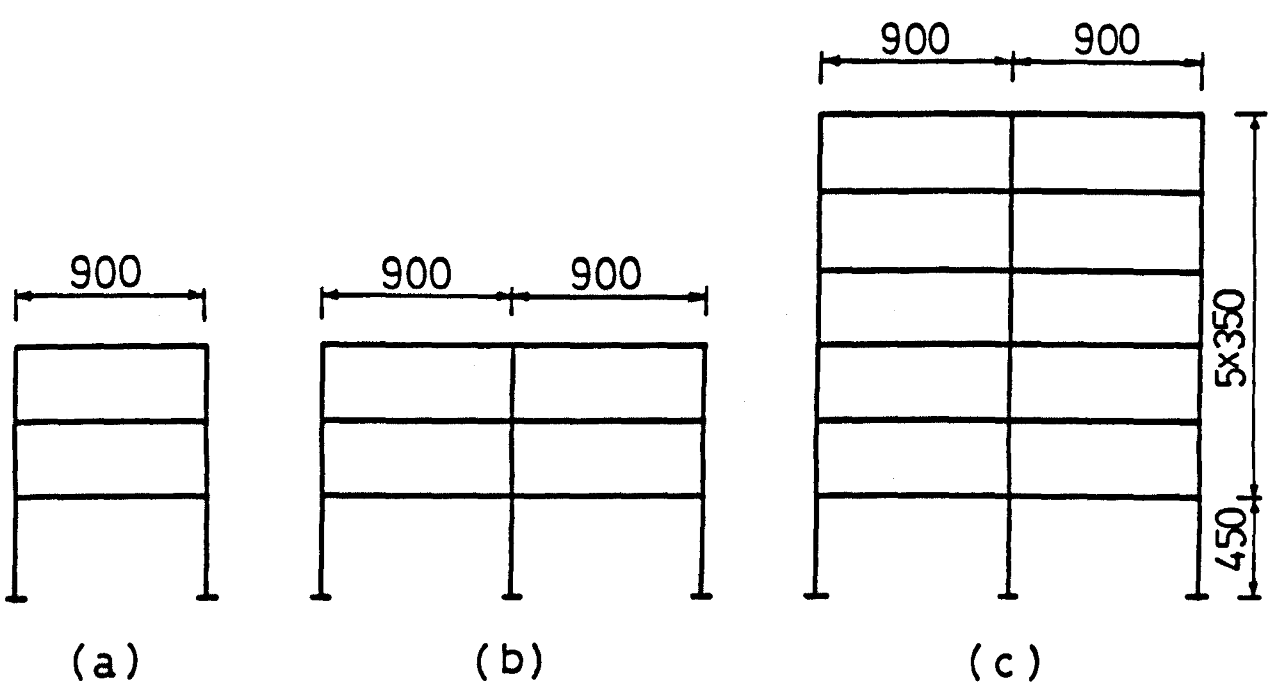


Fig. 3-14, Frames used in examples of approximate method
(a)3storey,1bay; (b)3storey,2bay; (c)6storey,2bay

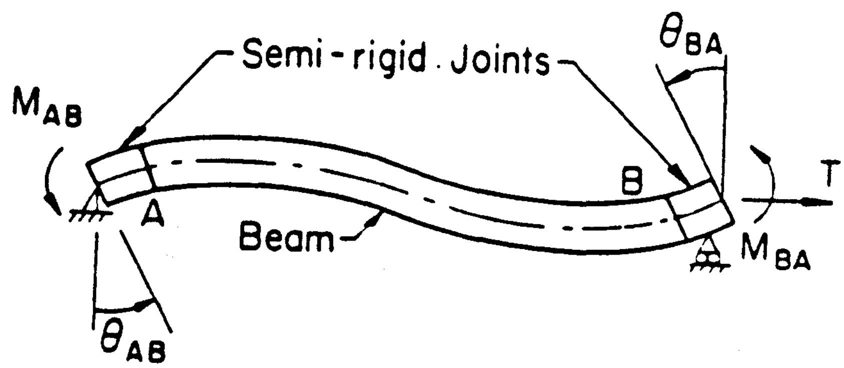


Fig. 3-15, Moments and rotations at the ends of a beam with semi-rigid joints

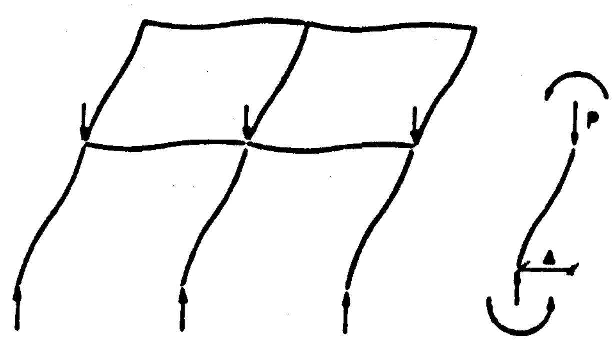


Fig. 3-16, Second-order effect

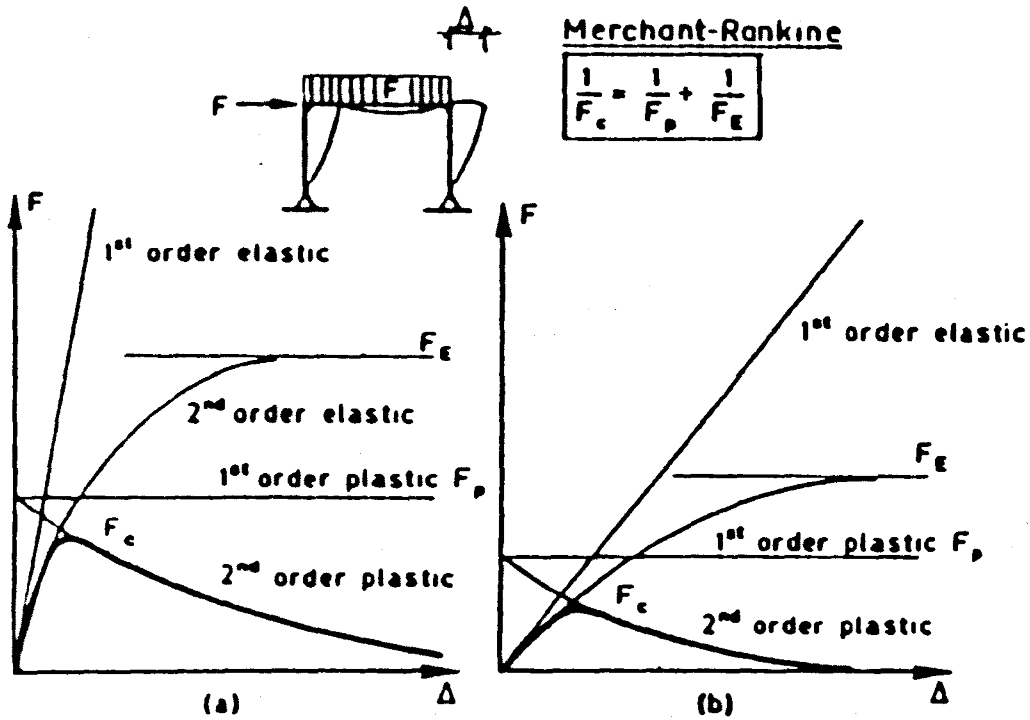


Fig. 3-17, Influence of semi-rigid and partial strength connections on the bearing capacity of an unbraced frame, (a)rigid and full strength connections, (b)semi-rigid and partial strength connections

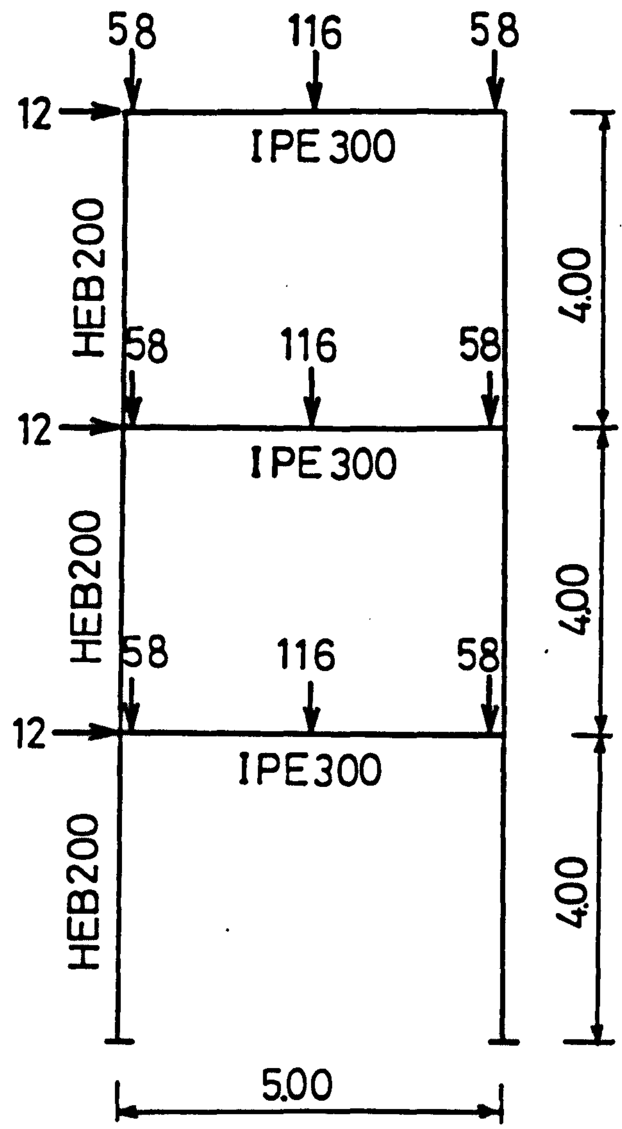


Fig. 3-18, Three storey,one bay frame used in the determination of collapse load levels

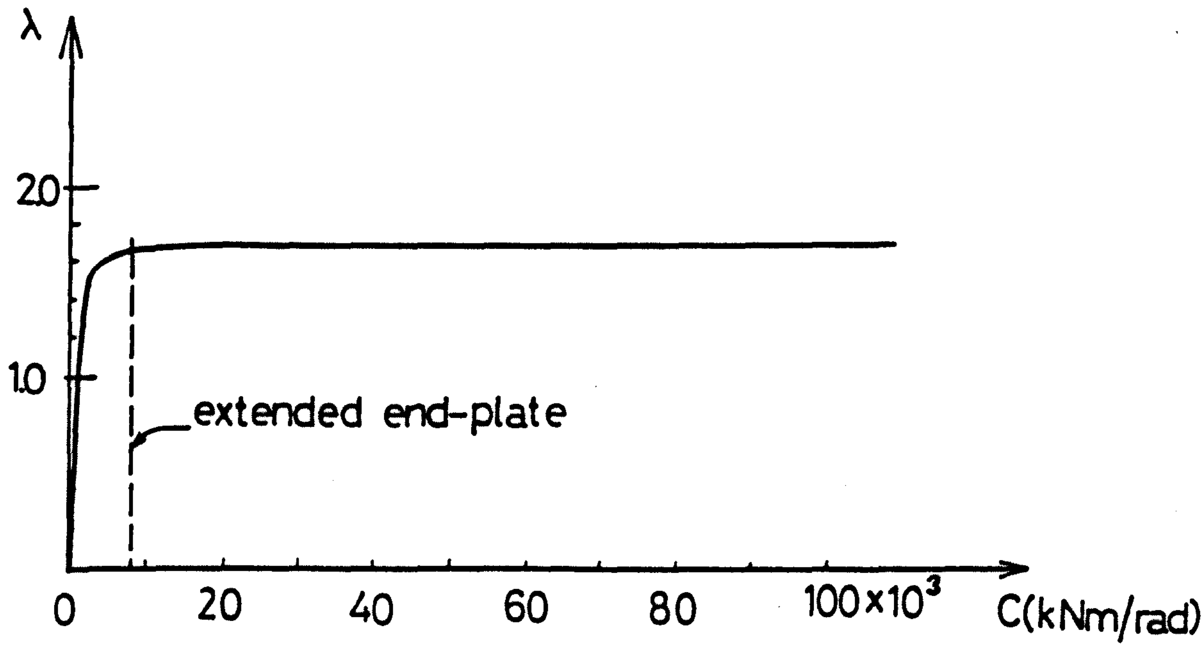


Fig. 3-19, Decrease in collapse load level due to joint flexibility

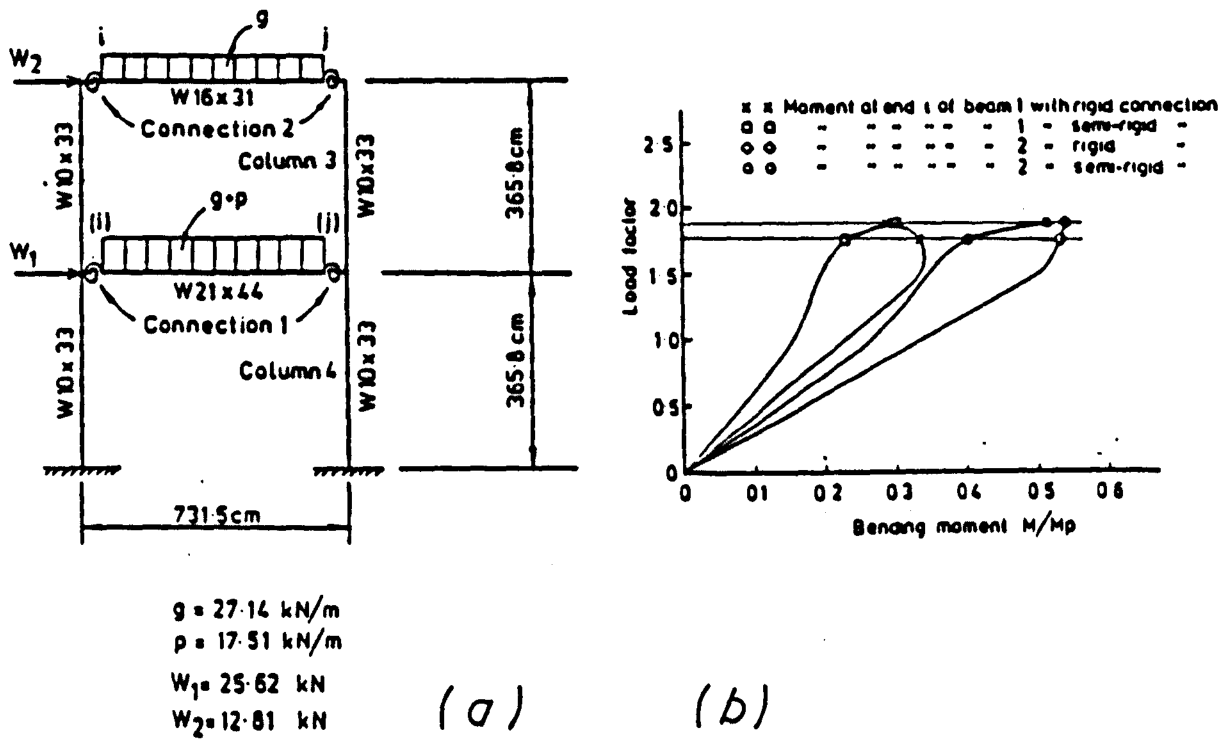


Fig. 3-20, Influence of semi-rigid joints on internal moments

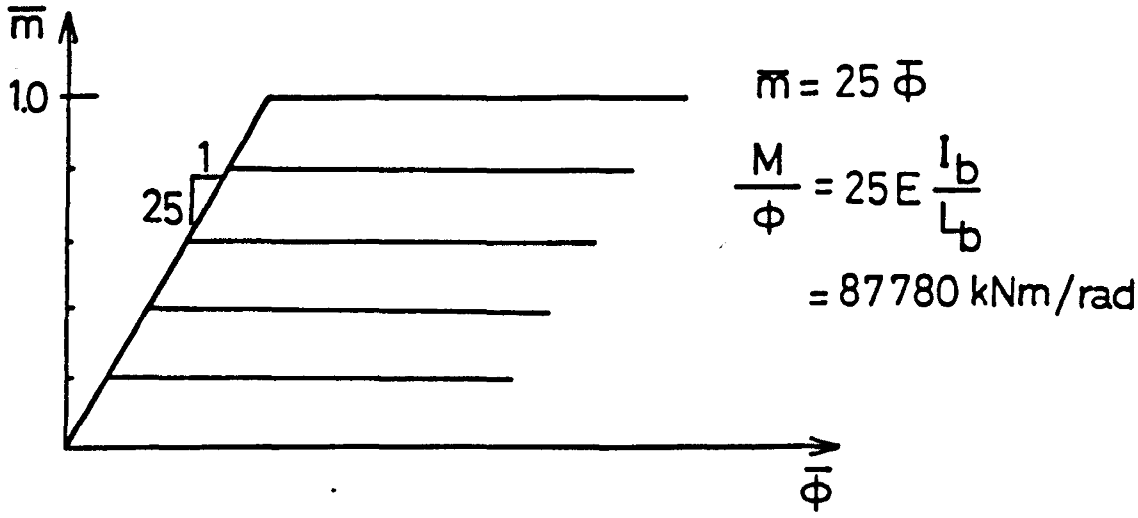


Fig. 3-21, Presentation of moment-rotation characteristics for partial strength connections used in the determination of collapse load levels

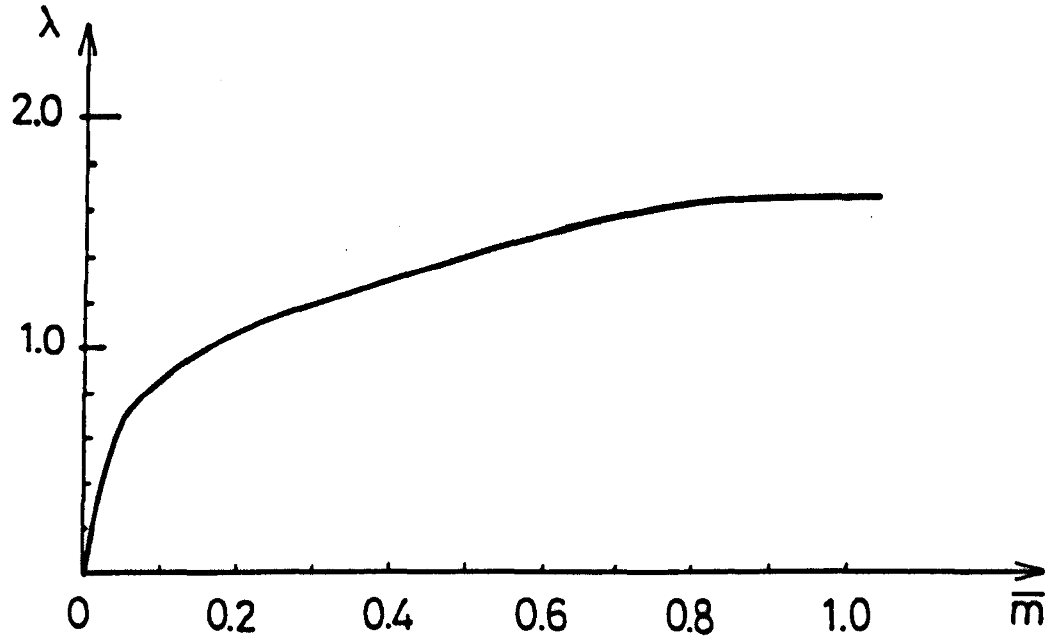


Fig. 3-22, Decrease in collapse load level due to partial strength joints

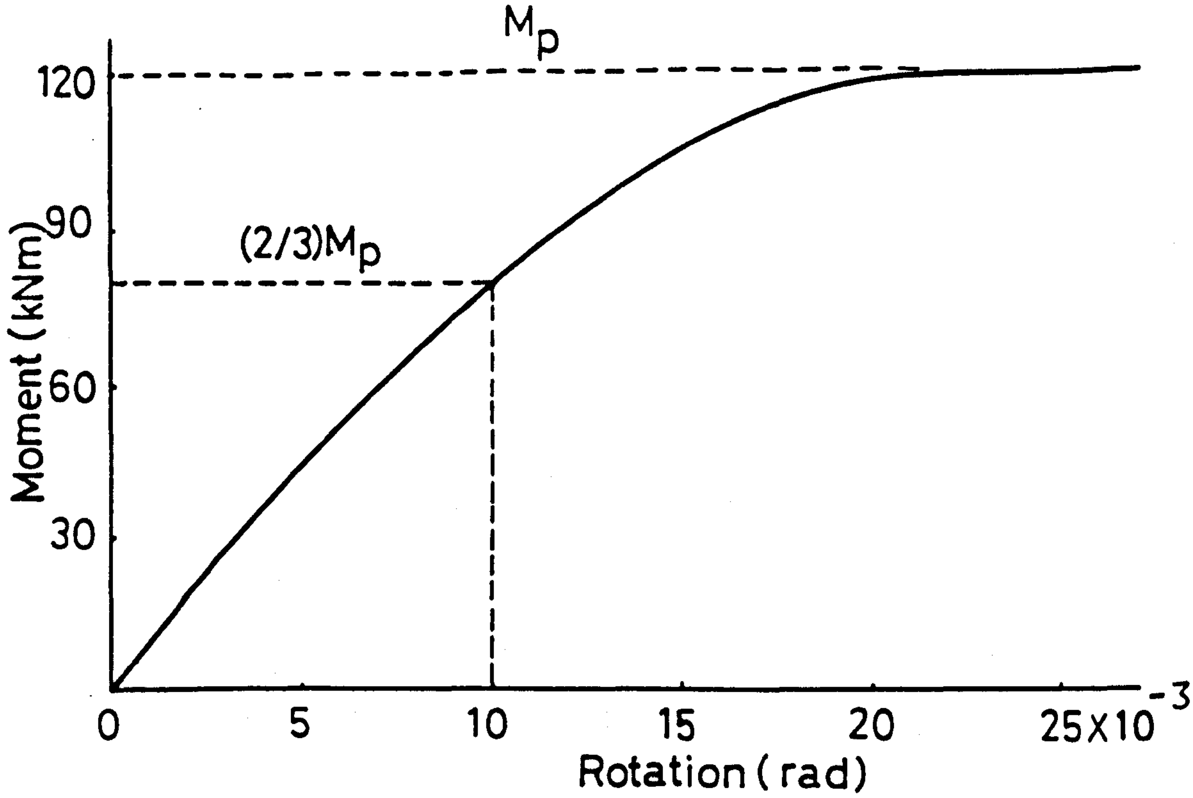


Fig. 3-23, The moment-rotation curve of an extended end-plate used in Fig. 3-19

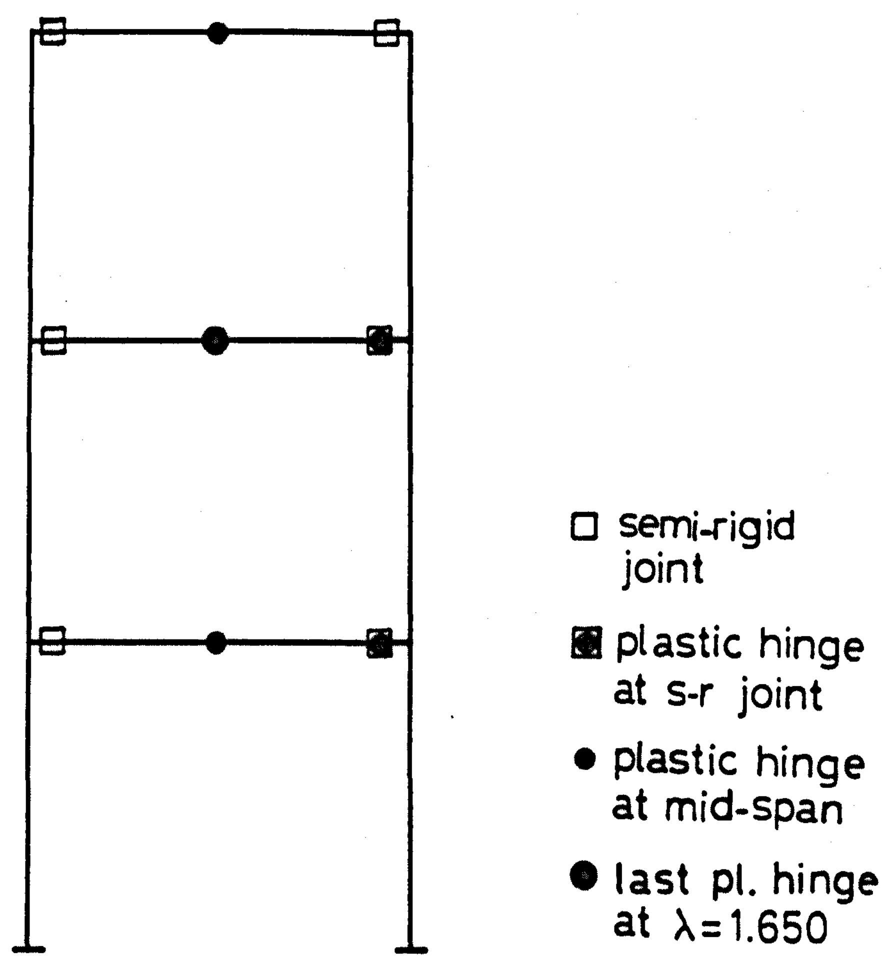


Fig. 3-24, Mode of collapse for $\bar{m}=0.8$

PART TWO

END PLATE CONNECTIONS

IN BRACED COMPOSITE FRAMES

Chapter 4
INTRODUCTION TO COMPOSITE CONNECTIONS

The demands for increasingly economical structures, which at the same time must possess adequate strength and stiffness, focus attention on the use of composite steel-concrete beams and columns in multistorey frames. The use of composite beams has been recognized by many national codes of practice for many years. BSI provided a specification for composite design first in 1959, AISC in 1963 and Canadian Standard Association in 1978.

The beams used in buildings are in reality members of framed structures, the behaviour of which is highly influenced by the detailing of the connections. Traditionally, the joints of steel framed structures were assumed to be fully rigid or completely flexible. New codes of practice including Eurocode 3(1992) have recognized the semi-rigid nature of connections, ranging from flexible to rigid dependent on the configuration and properties of the joints.

The cost effectiveness of semi-rigid construction is likely to be of potentially greater importance in the field of composite structures. This fact has been indicated by Johnson & Hope-Gill(1972) and Owens & Echeta(1981). The conventional design approach for composite beams is to assume that they are simply supported. Although ideal continuous beams may be made for bridges, the steel beams are jointed in a conventional frame, and continuity is obtained by an appropriate selection of longitudinal reinforcement in the vicinity of internal supports. This will substantially improve the frame's performance at serviceability and ultimate limit states. Fig. 4-1 shows the steelwork framing of a floor in a conventional composite frame.

As described in Chapter 1, the moment resistance in the hogging region of composite beams is normally smaller than that in the sagging region. A large amount of rotation capacity is then required of composite connections to achieve a full plastic mechanism in the composite beam. The two important factors in the behaviour of composite beams are therefore the moment resistance and the rotation capacity of the composite joints.

Attempts have been made in both experimental and theoretical studies to find these two main characteristics of composite connections. This chapter includes a historical survey that covers the experimental background. In chapter 5 experiments conducted by the author will be described. The results of these tests will be assessed in Chapter 6. Chapter 7 includes a brief literature survey of the theoretical background for the derivation of prediction equations for connection stiffness. The work undertaken by the author on this subject is explained and the proposed model is examined against the test data. Chapter 8 concerns studies on redistribution of moments in composite beams.

4-1-Definition of Composite Connection in EC4

Design that relates only to steelwork components within composite structures is not treated by Eurocode 4(1992). Hence in EC4 a composite connection is defined as " a connection between a composite member and any other member in which reinforcement is intended to contribute to the resistance of the connection". In the typical floor arrangement of Fig. 4-1, different types of beam-to-column or beam-to-girder connection configurations are present. Fig. 4-2 shows a typical composite connection between a composite beam and a steel column. The concrete slab may be composite or solid, i.e. with or without profiled steel sheeting. The shear connectors may be of different shapes; and welded, bolted or riveted to the steel section at the interface of concrete slab and beam. The most practical type of shear connector is the welded headed stud.

This definition will exclude connections with mesh reinforcement only, because concern over lack of ductility usually causes welded mesh to be omitted from the effective section. With the concrete cracked due to hogging bending and the profiled steel sheeting neglected, the connection reduces to one between steel sections and is therefore within the scope of EC3.

4-2-Definition of Moment Resistance and Rotation Capacity

The moment resistance of a composite connection is the maximum bending moment that can be applied on the actual connection, allowing for plastic deformations to take place. The design moment of the composite connection is the calculated moment resistance using a rational design method in which the partial safety factors are taken into account.

The design rotation capacity of a composite connection is the maximum amount of rotation that takes place at moments greater or equal to the design moment resistance of the connection.

4-3-Classification of Connections in EC4

EC4 adopts the classification of connections given in EC3, but with definitions of $M_{Pl.Rd}$ and EI_b appropriate to composite beams. This classification has been described in Chapter 3. Semi-rigid connections in sway frames are excluded from the scope of EC4. The boundaries for connections in braced frames are shown in Fig. 4-3.

The behaviour of semi-rigid connections in composite frames is a matter of current research. This research aims at introducing some criteria for determination of moment resistance, stiffness and rotation capacity of the connections. The following categories are intended for composite connections:

a) Simple connections: Any moment resistance and stiffness of the connection is ignored.

b) Rigid connections: No flexibility is assumed for the connection. The connection is usually full strength.

c) Semi-rigid connections: The moment resistance and the stiffness of the connection are taken into account. The connection may be full strength or partial strength.

Experimental and theoretical studies conducted by previous authors and the present author are available to clarify the classification of composite connections, as well as their main characteristics.

4-4-Experimental Background

An attempt to collect available experimental data on composite connections was made first by Zandonini(1989). These are covered in the historical background described here, together with tests performed elsewhere. Furthermore, the extensive experimental programmes undertaken recently by several researchers are summarized in this section.

Although several categories of composite connections are utilized in practice, as shown in Fig. 4-1, two types of them have been tested mainly. These are internal and external beam-to-column connections which are modelled as cruciform and cantilever specimens as shown in Fig. 4-4.

Different types of loading on cruciform specimens have been used by researchers as shown in Fig. 4-5. In the following paragraphs, the arrangement (a) of Fig. 4-5 with equal loads on both sides is referred to, unless otherwise stated. In the case of external connections, the load has been applied to the free end of cantilevers.

Early experimental studies on the behaviour of composite connections were undertaken by Proctor(1969) on encased beams with two types of cleated joints, one to column flange and one to the column web. The web connection had a much greater ultimate strength mainly resulting from the limited deformation of the web but also because of increased concrete cover on the tension cleat.

Further work on encased beams framed into exterior columns of various types was carried out at Liege by Dobruszkes et al(1969). In the first series, the cased beams were connected to reinforced concrete columns by means of various methods of anchorage. The second series were fully welded connections between encased beams and encased columns. The collapse load could be safely predicted from simple ultimate strength considerations. However, this kind of welded connection is rare in practice.

Subsequent tests, described below, are summarized in Table 4-1. In Table 4-2, comparison is made between the plastic resistance moment of the steel section, M_{pss}, negative plastic moment of composite beam, M_{pc}, the observed ultimate moment resistance of the joint, M_u, and the moment at which cracking of slab started, $M_{crac.}$. Table 4-2 also gives the ultimate observed rotations at failure and the modes of failure with which the experiments were terminated.

An experimental project was conducted by Daniels et al(1970) on fully welded composite beam-to-column joints as illustrated in Fig. 4-6(a). They performed two tests on interior joints of unbraced frames subjected to vertical and horizontal loads. Four regions in the beams of a floor were identified with regard to the type of bending moment distribution as shown in Fig. 4-7. Consequently, the configuration of applied loads on the specimens were variable to model the above four regions . There was no attempt in these tests to deduce the moment-rotation characteristics of the joints.

Barnard(1970) proposed taking advantage of semi-rigid composite action in design. The problem of local buckling in hogging moment regions had been appreciated earlier resulting in further studies on the behaviour of the compression zone in the beam section to provide an adequate rotation capacity in the connection. Climenhaga & Johnson (1972) pointed out that severe limitations have to be applied to the slenderness ratios of the web and the compression flange of composite beams. It is partly due to the raising of the plastic neutral axis resulting from the presence of slab reinforcement. The latter causes a greater portion of the web to undergo plastic deformation in compression and therefore influences the rotation capacity of the beam in a hogging moment region.

Johnson & Hope-Gill(1972) later conducted a series of tests on interior composite beam-to-column joints to study their behaviour as semi-rigid connections. They examined bottom flange cleat joints as shown in Fig. 4-6(b) with a cruciform configuration and symmetrical loading. They investigated the web slenderness of the steel section and defined a "force ratio" $A_r f_{yr}/A_s f_{ys}$ between the axial resistances of the reinforcement bars and the structural steel section. They concluded that the higher this ratio, the more critical web buckling. The rotation capacity of three rigid jointed beams was influenced by flange buckling, but in the beams with semi-rigid joints little flange buckling occured, due to the restraint provided by the angles and because prior failure took place in the rebars.

Ansourian(1974) tested rigid connections between steel I-beams and pin-ended concrete filled square hollow section columns. Two basic types of connection, welded and bolted, were used; but in both cases the compression component of the beam couple was transmitted by a welded flange plate and the beam shear being carried by a shear plate. The first type of specimens collapsed by the failure of the butt weld connecting the flange plate to the column face. The moment capacity of such a connection then was suggested to be determined by the strength of butt weld or the column wall, as had been concluded earlier by Valbert(1969). The second type of joints showed a satisfactory strength because of the concrete filling and the tension flange being connected to the back of the column.

Ansourian & Roderick(1976) carried out tests on exterior connections with encased columns, similar to Fig. 4-6(c), together with three tests on bare steel connections. Particular attention was paid to the slab reinforcement, to determine conditions under which it contributes to the effective composite action. The connections where the tension flange was not positively attached to the column flange had a satisfactory behaviour under working load, but premature collapse of the slab resulted. In comparison with bare steel, the composite connections were found to have double the moment resistance.

Ansourian(1977) conducted six more tests but on connections to interior columns (Fig. 4-6(c)) with symmetrical and asymmetrical configurations and loading. The columns were subjected to an axial force of 20% of the squash load. The slab reinforcement was identical in each test(2.2%), equal in area to 80% of the joist's cross sectional area. The variables were the dimensions of the joist and the type of loading. The tests were terminated by local buckling of compression flange, accompanied by lateral displacement in one case. The extent of yielding before buckling was found to be greatest for low values of the flange width to thickness ratio (B/T).

More tests on different types of composite connections were performed by Ansourian & Sase(1981) as illustrated in Figs. 4-6(d)and(e). A total of five specimens were tested, three with external and two with internal encased columns. The first used flush end-plates; the second, fourth and fifth utilized top and bottom flange cleats; and the third adopted a flush end-plate connection with backing plates. The flexibility of the cleated joint allowed larger deflections to develop in comparison with the above mentioned (1977) tests. In the interior column specimens, the full plastic moment was exceeded by 15% and failure occurred by local buckling of the steel section when sufficient of the flange and web were plastified to allow a full wavelength of the buckle to develop . At external connections, the maximum moments did not reach the plastic moment before brittle collapse of the slab occurred. The strength and stiffness of the connections were found to be greatly enhanced by the column casing and the reinforcing of the concrete slab. The transfer of moments was of the same order as in similar test specimens having welded flange plate connection and the same reinforcement layout.

Tests by Echeta & Owens(1981) aimed to check the feasibility of meeting the requirements of a novel plastic design approach proposed for semi-rigid composite frames. They tested five specimens (Echeta(1982)), one on an interior connection and the rest on exterior connections. Different types of joints were used (Figs. 4-

6(f),(l)and(m)) with very low slab reinforcement ratios as a consequence of the limited moment resistance required by the proposed design method. The loading arrangement was as in Fig. 4-5(b). The relative values of P_1 and P_2 were controlled so that the shear to moment ratio at the joint zone was varied while the moment at the column face was kept constant. This was to simulate changes in this ratio when the moment redistribution was occurring in the beam as the collapse approached. No adverse interaction between high shear force and high moment was found. They concluded that the rotation capacity of the connection is large enough to allow a high degree of moment redistribution without buckling of the beam flange or the web occurring. They also examined the effect of lack of fit between the column face and the beam, and observed that it would reduce the connection stiffness and increase crack widths in the concrete slab, if premature slip occurred at the bottom cleat.

Johnson & Law(1981), Law(1983) carried out three groups of full scale tests each including two cruciform specimens. The first two were flush end-plate connections to column flanges, similar to Fig. 4-6(e), and the next two to the column web (Fig. 4-6(j)). One column of each group was encased and the interaction between steel and concrete in the vicinity of the column was different. The axial force in the column was provided by post-tensioning four prestressing bars. The stud arrangements on either side of the column also differed. On one side of all these four specimens the usual practice of equal spacing was used (full interaction) and on the other beam the shear connectors were bunched away from the column (partial interaction). The details of the third group of specimens were similar to the first group, except that the shear connectors were uniformly spread along the beams on both sides. The only difference between the two specimens of this group was the slab thickness (200 mm compared to 125 mm), whilst the amount of the slab reinforcement was equal. Therefore the reinforcement ratio was 0.63% in comparison with 1.0% for the others. An analytical method was suggested, which provided a conservative formula for predicting the elastic stiffness of composite beam-to-column connections with flush end-plate steel joints. This method will be described in Chapter 7.

In order to simulate chequerboard loading conditions, the tests were conducted by applying first the load on one side only and increasing it up to the attainment of the plastic resistance of the relevant joint. The load on the other side was then applied and increased while the load on first side was kept constant until both loads were equal. Finally the both loads were increased together up to collapse.

Van Dalen & Godoy(1982) conducted five tests on different types of interior connections representing the flexible, rigid and semi-rigid joints as shown in Figs. 4-6(d),(g) and (h). Comparison was possible with similar bare steel connections as three additional tests on steel connections were undertaken . The loading was symmetrical. The reinforcement ratio changed from 0.46% to 0.80%, which was considered to bound the range of practical interest. The increase in moment resistance that resulted from the increase in the percentage of slab reinforcement was accompanied by significantly smaller rotations. Slippage occurred in the flexible connection at early stage of loading. This suggests that other types of connections such as welded cleats to the beam or flush end-plates would be preferable. The results showed that an increase in the amount of longitudinal reinforcement resulted in a greater increase in moment resistance for a flexible beam-column connection than in a semi-rigid one. The seat angle used in all tests served to stiffen the bottom flange of the beam. This in turn could cause an adventageous increase in the moment resistance of the composite beam. Stiffened columns were used in all tests. However, seat angles might also benefit the unstiffened column. The ultimate moment resistance of the composite connection was found to be six times that for the relevant bare steel case for a flexible joint and 1.5 times for the rigid joint.

Two cruciform full scale sub-assemblages were tested by Redwood et al(1985) to investigate the effect of fully reversed cyclic loading on the behaviour of connections.

The only difference between the two specimens was the loading conditions as shown in Figs. 4-5(e)and(f). These two conditions cause both negative and positive bending moment at the column face. The resistance of the first specimen exceeded the negative plastic moment measured under monotonic loading. The second specimen demonstrated the stability and high energy absorbing capacity of the column web panel zone. Despite the crushing of concrete at the column face, the positive moment resistance of the column was not reached and the resistance equalled that measured under monotonic loading. In the both cases considerable ductility was achieved and the tests were terminated because of lateral buckling which accompanied local buckling in the case of the first specimen . The similarity between the moment-rotation envelopes of monotonic and cyclic loading was evident.

A research programme has been carried out at the University of Minnesota (Leon & Ammerman(1986), Ammerman & Leon(1987) and Leon(1987)) in order to check the suitability of semi-rigid composite joints for sway frames. The first phase of this project was intended to provide data to compare the behaviour of a semi-rigid connection with and without a concrete slab. A cleated steelwork connection(see Fig. 4-6(k)) similar to those of Radziminski et al(1982)(1985) was used for which the M-ϕ data was already provided. Two composite interior beam-to-column connections were tested. The first was subjected to monotonic loading and the second to cyclic loading. The test set-up for both was similar except that in the second test a lateral load was applied at the bottom of the column instead of vertical load at the beam ends(see Fig. 4-5(d)). The only difference in connection details was that the top flange angle was omitted on one side of the monotonic test and on both sides of cyclic test specimen. The two different steel connections in the same specimen were designed to check the influence of the degree of continuity given by the top flange cleat. They concluded that the behaviour of a composite connection is similar to that of a non-composite one with the slab reinforcement replacing the top angle. The substitution of the angle with rebars resulted in much more linear initial behaviour. The degree of linearity was expected to become more if the reinforcement ratio was increased. Although the column web was not stiffened, only very limited yielding was observed. It was then concluded that web crippling might not be as severe a problem for semi-rigid connections as with rigid ones. The results of the first phase of their work was extended to a full scale test on a two-bay frame and eventually into practical applications.

The second phase of this programme has been reported by Leon(1990). It included tests on four types of cleated joints subjected to monotonic or cyclic loading. Slippage was observed at the interface of cleats and the steel beam. The drift of the specimen was monitored in the cyclic tests, and a thicker bottom cleat was found to improve the hysteric behaviour of the joint. It was concluded that a semi-rigid composite system would be very advantageous in the areas of moderate to high wind loads, and low to moderate seismic loads. A design procedure was then developed from the understanding of the failure mechanisms for these connections.

A research project was carried out by Benussi et al(1986)(1987) in which end-plate connections were employed as the most suitable joints. Two factors were investigated: the strength of the slab reinforcement and the type of the joint. The test procedure has been reported in two parts. The first series consisted of four cruciform connections, two flush end-plate as semi-rigid connections(Fig. 4-6(e)) and two header-plate as flexible connections(Fig. 4-6(i)). In the latter the plate was simply welded to the lower part of the web. The only difference in each couple of specimens was the percentage of rebars. Despite the remarkable difference of stiffness and strength, the shape of $M-\phi$ curves were basically similar for all specimens. They concluded that a general graph as shown in Fig. 4-8 could be assumed to characterize the elastic, inelastic and plastic phases. Consequently, a quadrilinear relationship was suggested. It was found adviseable to improve the capabilities of the joint by properly selecting the amount and tensile strength of the rebars rather than by a more complex detailing of the steel connection. In a subsequent numerical study on braced frames, the amount of reinforcement was also found to affect the behaviour of beams with semi-rigid joints more than that of beams with fixed ends. In this study the use of simplified models for the $M-\phi$ relationships such as the proposed one, resulted in the development of plastic hinges first in the end cross section.

The second series of their work was conducted to point out the influence of shear connector flexibility, the interaction between the concrete slab and the column, and the effect of unbalanced moments. The first connection tested was as Fig. 4-6(i) but the column web stiffener was omitted and the studs were of a smaller size and spaced at a greater interval, while still considered as providing full shear interaction. In four of the other five specimens a gap was left between the slab and the left side of both column flanges in order to allow contact solely along their right side. In two of these tests, mechanically fastened shear connectors were used instead of the welded studs, while the number and spacing was kept the same. The last specimen was a fully welded connection to an unstiffened column. In the assessment of the results, elastic phase was divided into cracked and uncracked stages. In the inelastic phase, further increments of moment resistance were made possible mainly by strain hardening of the rebars. In this phase the specimens with the mechanically fastened shear connectors experienced a significant uplift of the slab with a modest influence on the overall response. The tests were stopped because of excessive deformation before any actual evidence of collapse was observed. The gap of concrete at the column face influenced the response of one side significantly in the inelastic range because of the absence of the slab-column contact.

The measured rotation at a distance from column face including the curvature of the relevant part of the beam, and the vertical deflections of the beams end were used in two alternative approaches to calculate the rotation at the joint. It was suggested that the internal beam-column interaction be modelled as three non-linear springs. An "interaction coefficient" was introduced with reference to the quota of moment transmitted between the two adjacent beams. This coefficient ranges from 0 to 1, the former for no joint interaction and the latter for full joint interaction. The ultimate moment resistance achieved by the costly fully welded connection was limited whilst the rotation capacity reduced.

Recently, an extensive experimental study was conducted at the University of Liege by Altman et al(1990). The behaviour of 24 beam-to-column web and/or flange cleat connections bolted to column flanges similar to Fig. 4-6(f) was investigated. The variable parameters were the thickness of cleats, number of cleats, reinforcement, testing arrangement and instrumentation. The influence of each parameter together with experimental curves has been briefly described in their preliminary report, which has yet to be ellaborated.

A series of tests on five external and six internal connections has been carried out by Lam(1989). Two tests of each group was on minor axis joints. An additional beam-to-beam connection was also tested. Seven bare steel joints had been tested prior to the tests on the composite connections. Hence comparison was possible between the behaviour of steelwork and composite connections. The aim was to study the effect of deck direction, column orientation, internal or external column position and slab reinforcement. The connection type was constant. It comprised a seating cleat and a single side cleat to the beam web. In cruciform tests, load was applied on top of the column. The end of the cantilever beams then reacted against the rig.

The stiffness of uncracked slab was found to be significantly greater when the deck was parallel to the beam. Only modest rotation capacity was observed in tests with nominal reinforcement, due to brittleness of mesh. Additional reinforcement improved both the rotation capacity and the moment resistance of the connections. The analytical study undertaken demonstrated that the moment-rotation characteristics of composite joints may improve the load resistance of composite beams and reduce service deflections.

A comprehensive programme of testing and analytical work has begun at the University of Nottingham, with two aims. The first project concerns collecting the available test results, conducting a series of further tests on composite joints and modelling the connection behaviour. The second project concentrates on the effect of semi-rigid composite joints on the behaviour of frames, particularly the redistribution of moments along the continuous beams. A number of full scale tests on composite frames are also included in this project.

Regarding the first project, sixteen tests have been performed so far (Xiao et al(1990)(1991)(1992)) on different types of joints with various amount of reinforcement but mainly with identical beam and column sections. Four connection types have been considered: flush end plate, partial depth end plate, seating cleat and web side plate as shown in Figs. 4-6(e),(i),(d) and (n). They were bolted to either the column web or the flange (minor and major axis joints), except for the web side plates which were welded to the column and bolted to the beam web. The main results of this series of tests are the moment-rotation characteristic of the connections.

A test programme is being conducted by Jarrett & Moore(1992) on full scale H-frames, two of which consisting of a composite beam jointed to two columns via semi-rigid connections as external joints. The third subframe consisted of three columns and two beams providing an internal and two external joints. The first test employed web side plate connections similar to Fig. 4-6(n) and absorbed less than 15% of the midspan moment, despite a heavily reinforced slab(1.5%). The second and the third tests comprised partial depth end plate connections absorbing 30% and 25% of the midspan moment respectively. The results of this series of tests has not yet been published.

Table 4-1, Summary of tests on composite connections

AUTHOR & YEAR	REFERENCE	ARRANGEMENT	CONN. TYPE	COLUMN SECTION	BEAM SECTION	f_{ys} (N/mm^2)	SLAB W/D (mm/mm)	REINF. (%)	f_{yr} (N/mm^2)	LOAD TYPE	COMMENTS
Daniels,											
Kroll &	J1	INT-S	a	16WF68	16WF40	278	1600/100	1.1	-	d	
Fisher	J2	INT-S	a	16WF68	16WF40	278	1600/100	1.8	-	d	
1970											
Johnson	HB50	INT-S	b	152x152x37	203x133x25	310	760/83	1.4	363	a	
&	HB51	INT-S	b	152x152x37	305x165x40	277	760/90	0.8	390	a	
Hope-Gill	HB52	INT-S	b	152x152x37	305x165x40	277	760/90	1.8	404	a	
1972	HB53	INT-S	b	152x152x37	305x165x40	293	760/90	2.0	402	a	
	HB54	INT-S	b	152x152x37	406x140x39	315	760/90	2.0	391	a	
Ansourian	RSC1	EXT-S	c	150x125x38	PL100x75x15	288	600/75	2.2	270	-	all columns
&	RSC2	EXT-S	c	150x125x38	PL150x75x18	346	600/75	2.2	285	-	were encased
Roderick	USC1	EXT-S	c	200UC46	200UB25	288	600/100	3.6	286	-	
1976	USC2,3,4	EXT-S	c	200UC46	200UB25	290	1200/100	2.1	280	-	bars configs.
	USC5	EXT-S	c	200UC60	200UB25	287	1200/100	2.1	300	-	were different
Ansourian	1	INT-S	c	200UC60	200UB25	295	1200/100	2.2	271	a	all columns
1977	2	INT-S	c	200UC60	200UB30	275	1200/100	2.2	305	c	were encased
	3,4,5,6	INT-S	c	200UC60	200UB30	276	1200/100	2.2	300	c*	*loads differed
Ansourian	1	EXT-S	e	200UC46	200UB25	335	1200/100	2.1	316	-	all columns
&	2	EXT-S	d	200UC46	200UB25	335	1200/100	2.1	316	-	were encased
Sase	3	EXT-S	e*	200UC46	200UB25	335	1200/100	2.1	316	-	*back plates
1981	4	INT-S	d	200UC46	200UB25	326	1200/100	2.1	315	a	
	5	INT-S	d	200UC46	200UB25	326	1200/100	2.1	315	c	
Echeta	1B	INT-S	f	150x100x5*	254x102x28	311	1050/75	0.5	540	b	*RHS filled
&	2BS	EXT-S	l	150x100x5*	254x102x28	311	1050/75	0.4	415	b	
Owens	3,4BS	EXT-S	m	150x100x5*	254x102x28	462	1050/75	0.4	555	b	
1981	5BS	EXT-S	m	150x100x5*	152x152x37	378	1050/75	0.4	555	b	
Johnson	JX1	INT-S	e	203x203x46	457x191x67	264	1400/125	1.0	461	a	
&	JX2	INT-S	e	203x203x46	457x191x67	264	1400/125	1.0	461	a	column encased
Law	JY1	INT-W	j	254x254x73	457x191x67	264	1400/125	1.0	461	a	
1981	JY2	INT-W	j	254x254x73	457x191x67	264	1400/125	1.0	461	a	column encased
	JC1	INT-S	e	203x203x46	457x191x67	264	1400/125	1.0	461	c	column encased
	JC2	INT-S	e	203x203x46	457x191x67	264	1400/200	0.6	461	c	column encased

Table 4-1, Continued

AUTHOR & YEAR	REF-ERENCE	ARRANGE-MENT	CONN. TYPE	COLUMN SECTION	BEAM SECTION	f_{ys} (N/mm^2)	SLAB W/D (mm/mm)	REINF. (%)	f_{yr} (N/mm^2)	LOAD TYPE	COMMENTS
Van Dalen	CB1	INT-S	d	W8x20	W8x20	310	1200/100	0.5	483	a	
&	CB2,3	INT-S	g	W8x20	W8x20	310	1200/100	0.8	483	a	
Godoy	CB4	INT-S	h	W8x20	W8x20	310	1200/100	0.5	483	a	
1982	CB5	INT-S	h	W8x20	W8x20	310	1200/100	0.8	483	a	
Redwood,											
Mitchell	1	INT-S	a*	W250x89	W360x39	-	1750/132	1.6	-	e	transv. beams
&Dunberry	2	INT-S	a*	W250x89	W360x39	-	1750/132	1.6	-	f	*no stiffeners
1985											
Benussi,	SJA-10	INT-S	i	HEB260	IPE300	288	1000/120	0.7	495	a	
Puhali &	SJA-14	INT-S	i	HEB260	IPE300	288	1000/120	1.2	413	a	
Zandonini	SJB-10	INT-s	e	HEB260	IPE300	288	1000/120	0.7	495	a	
1986	SJB-14	INT-S	e	HEB260	IPE300	288	1000/120	1.2	413	a	
Ammerman	SRCC1M	INT-S	k	WF14x99	WF14x38	293	1500/100	0.7	435	a	
& Leon	SRCC1C	INT-S	k*	WF14x99	WF14x38	293	1500/100	0.7	435	d	*top flange
1987											
Puhali,	SJA14/1,2	INT-S	i	HEB260	IPE300	316	1000/120	1.2	421	a	studs
Benussi	SJA14/3	INT-S	i	HEB260	IPE300	316	1000/120	1.2	421	c	studs
&	SJA14/4	INT-S	i	HEB260	IPE300	316	1000/120	1.2	421	a	connectors
Zandonini	SJA14/5	INT-S	i	HEB260	IPE300	316	1000/120	1.2	421	a	connectors
1989	SJ14	INT-S	i	HEB260	IPE300	316	1000/120	1.2	421	a	studs
Altman,	24x2C	INT-S	k*	HE200	IPE240	-	1200/120	%	-	a	not reported
Maquoi	30x2C	INT-S	k*	HE200	IPE300	-	1200/120	%	-	a	*k:right side
&	36x2C	INT-S	k*	HE200	IPE360	-	1200/120	%	-	a	%0.7,1.3,2.1
Jaspart	24x3C	INT-S	k**	HE200	IPE240	-	1200/120	%	-	a	not reported
1990	30x3C	INT-S	k**	HE200	IPE300	-	1200/120	%	-	a	**k:left side
	36x3C	INT-S	k**	HE200	IPE360	-	1200/120	%	-	a	totally 24 tests
Leon	SRCC3C	INT-S	k	W14x120	W14x38	293	1500/125	0.9	435	d	
1990	SRCC4M	INT-S	k*	W14x145	W21x57	293	1500/125	0.9	435	a	*seat plate
	SRCC5M	INT-S	k**	W14x145	W21x57	293	1500/125	0.9	435	a	**web cleat
	SRCC6C	INT-S	k**	W14x145	W21x57	293	1500/125	0.9	435	a	omitted

Table 4-1, Continued

AUTHOR & YEAR	REF-ERENCE	ARRANGE-MENT	CONN. TYPE	COLUMN SECTION	BEAM SECTION	f_{ys} (N/mm^2)	SLAB W/D (mm/mm)	REINF. (%)	f_{yr} (N/mm^2)	LOAD TYPE	COMMENTS
Davison,	C1	EXT-S	f	203x203x46	305x165x46	-	1800/120	0.2	-	a	one side
Lam	C2	EXT-S	f	203x203x46	305x165x46	-	1800/120	0.6	-	a	cleat
&	C3	EXT-S	f	203x203x46	356x171x67	-	1200/120	0.2	-	a	connections,
Nethercot	C4	EXT-W	f	203x203x46	356x171x67	-	1200/120	0.2	-	a	transverse
1990	C5	EXT-W	f	203x203x46	356x171x67	-	1200/120	0.9	-	a	girders
	C6	INT-S	f	203x203x46	305x165x46	-	1800/120	0.2	-	a	provided
	C7	INT-S	f	203x203x46	305x165x46	-	1800/120	0.6	-	a	in all
	C8	INT-S	f	203x203x46	305x165x46	-	1800/120	0.9	-	a	tests
	C9	INT-S	f	203x203x46	305x165x46	-	1800/120	1.7	-	a	
	C10	INT-W	f	203x203x46	356x171x67	-	1200/120	0.2	-	a	
	C11	INT-W	f	203x203x46	356x171x67	-	1200/120	1.7	-	a	
Bernuzzi,	CT1	INT-S	e*	HEB260	IPE330	329	1000/120	1.1	483	a	*extended in
Noe &	CT2	INT-S	e*	HEB260	IPE330	319	1000/120	1.1	473	a	compression
Zandonini	CT3	INT-S	e*	Tubular**	IPE330	312	1000/120	1.1	478	a	**filled
(1991)											
Xiao,	SCJ1	INT-S	d*	203x203x52	305x165x40	293	1200/120	0.2	558	a	*top cleat
Nethercot	SCJ2	INT-S	n	203x203x52	305x165x40	293	1200/120	0.2	558	a	omitted
&	SCJ3	INT-S	e	203x203x52	305x165x40	293	1200/120	0.2	558	a	
Choo	SCJ4	INT-S	e	203x203x52	305x165x40	293	1200/120	1.0	544	a	
1992	SCJ5	INT-S	e	203x203x52	305x165x40	293	1200/120	1.0	544	a	
	SCJ9	INT-S	i	203x203x46	305x165x40	293	1200/120	0.8	454	a	
	SCJ10	INT-S	i	203x203x46	305x165x40	293	1200/120	0.8	454	a	
	SCJ11	INT-S	d*	203x203x46	305x165x40	293	1200/120	0.7	454	a	
	SCJ12	INT-S	n	203x203x46	305x165x40	293	1200/120	0.7	454	a	

Note- 0.2% reinforcement in Table 4-1 refers to A142 mesh only.

Table 4-2, Moments, rotation capacities and modes of failure in previous tests

AUTHOR & YEAR	REF-ERENCE	M_{pss} (kNm)	M_{pc} (kNm)	M_u (kNm)	$M_{crac.}$ (kNm)	$\frac{M_u}{M_{pc}}$	ϕ_u (mrad)	FAILURE MODE
Johnson	HB50	81	120	101	35	0.84	>60	B
&	HB51	187	226	105	37	0.47	>70	A
Hope-Gill	HB52	184	249	222	78	0.89	>65	A
1972	HB53	175	285	254	89	0.89	30	C
	HB54	225	339	303	106	0.89	33	D
Ansourian	RSC1	-	35	34	16	0.98	34*	D
&	RSC2	-	65	62	20	0.96	54	L
Roderick	USC1	77	123	121	28	0.99	64	D
1976	USC2	77	123	115	40	0.93	61	F
	USC3	77	131	153	40	1.16	61	D
	USC4	77	131	106	40	0.81	41	F
	USC5	77	132	149	40	1.13	51	D
Ansourian	1	77	131	155	40	1.18	34*	D
1977	2	86	146	202	70	1.39	64	D
	3	195	295	319	52	1.08	30	D,L
	4	86	146	209	65	1.43	53	D
	5	86	146	191	52	1.31	45	D
	6	195	266	191	52	0.72	110	A
Ansourian	1	77	143	117	40	0.82	95	F
&	2	77	143	118	40	0.83	45	F
Sase	3	77	143	127	40	0.89	110	F
1981	4	77	136	161	-	1.18	-	D
	5	77	136	156	-	1.15	-	D
Echeta	1B	105	139	111	-	0.80	>32	A
&	2BS	105	126	65	-	0.52	>34	F
Owens	3BS	157	185	72	-	0.39	>30	A
1981	4BS	157	185	68	-	0.37	>50	G
	5BS	110	130	50	-	0.38	>45	A
Johnson	JX1	409	541	354	-	0.66	24	A
&	JX2	409	541	370	-	0.68	35	A
Law	JY1	409	541	384	-	0.71	10	A
1981	JY2	409	541	600	-	1.11	88	D
	JC1	411	541	449	-	0.83	19	F
	JC2	411	564	530	-	0.94	18	F
Van Dalen	CB1	95	123	121	27	0.98	47	A
&	CB2	95	142	173	27	1.15	36	A
Godoy	CB3	95	142	171	27	1.21	10	A
1982	CB4	95	123	138	27	1.12	22	A
	CB5	95	142	162	27	1.14	14	A
Redwood	1	187	337	380	84	1.13	-	L,D
et al. 1985	2	187	337	350	80	1.04	-	D
Benussi,	SJA-10	181	248	165	-	0.67	>21	A
Puhali &	SJA-14	181	273	221	-	0.81	>23	A
Zandonini	SJB-10	181	248	208	-	0.84	>22	A
1986	SJB-14	181	273	261	-	0.96	24	D

Table 4-2, Continued

AUTHOR & YEAR	REF-ERENCE	M_{pss} (kNm)	M_{pc} (kNm)	M_u (kNm)	$M_{crac.}$ (kNm)	$\frac{M_u}{M_{pc}}$	ϕ_u (mrad)	FAILURE MODE
Ammerman & Leon 1987	SRCC1ML	295	392	235	-	0.60	39	A
	SRCC1MR	295	392	305	-	0.78	47	E
	SRCC1C	295	392	235	-	0.60	-	A
Puhali, Benussi & Zandonini 1989	SJA14/1	181	285	246	61	0.86	27	A
	SJA14/2	181	285	242	47	0.85	40	A
	SJA14/3	181	285	230	-	0.81	37	A
	SJA14/4	181	285	240	47	0.84	48	A
	SJA14/5	181	285	220	-	0.77	23	A
	RJ14	181	285	287	47	1.01	13	D
Leon 1990	SRCC4M	588	-	570	94	-	12	F
	SRCC5M	588	-	423	-	-	39	A
	SRCC6C	588	-	194	59	-	8	E
Davison, Lam & Nethercot 1990	C1	198	-	30	8.5	-	27	G
	C2	198	-	50	14	-	26.5	G
	C3	333	-	60	18	-	21.5	G
	C4	333	-	14	4	-	28.0	G
	C5	333	-	94	20	-	27.0	G
	C6	198	-	32	16	-	26.0	-
	C7	198	-	140	-	-	38.0	-
	C8	198	-	181	19	-	38.0	-
	C9	198	-	160	40	-	28.0	F
	C10	333	-	175	30	-	23.0	A
	C11	333	-	200	32	-	12.0	-
Bernuzzi, Noe & Zandonini (1991)	CT1	252	372	298	53	0.80	31	C
	CT2	252	363	356	55	0.98	13	C
	CT3	252	361	300	55	0.83	11	C
Xiao, Nethercot & Choo 1992	SCJ1	172	220	49	22	0.22	14	F
	SCJ2	172	220	30	12	0.14	21	F
	SCJ3	172	220	86	30	0.39	7	F
	SCJ4	172	292	203	40	0.69	23	H
	SCJ5	172	292	241	45	0.82	30	D
	SCJ9	172	277	108	27	0.39	18	H
	SCJ10	172	277	148	38	0.53	17	D
	SCJ11	172	269	170	45	0.63	28	H
	SCJ12	172	269	101	22	0.38	44	D
	SCJ13	-	-	181	34	-	19	A
	SCJ14	-	-	83	20	-	31	D
	SCJ15	-	-	186	38	-	23	A
	SCJ16	-	-	225	30	-	-	-

A - test terminated for excessive joint deformation
B - failure of the shear connectors
C - failure of slab in shear
D - local buckling of the steel beam (flange and/or web)
E - shear fracture of the bolts
F - fracture of slab reinforcement
G - crushing of slab against the column
H - buckling of column web in compression
L - lateral buckling of beam

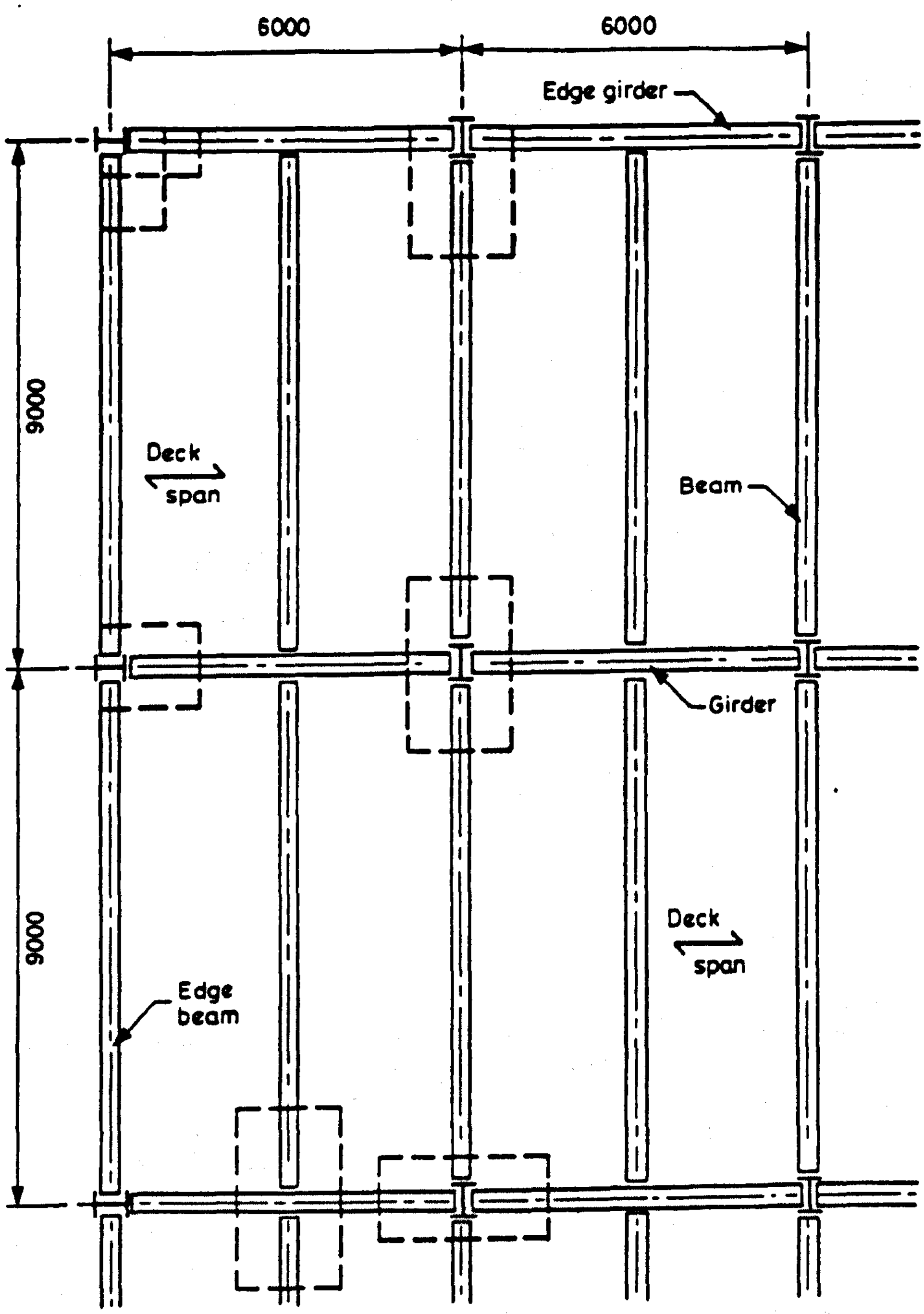


Fig. 4-1, Steelwork framing to support a composite floor

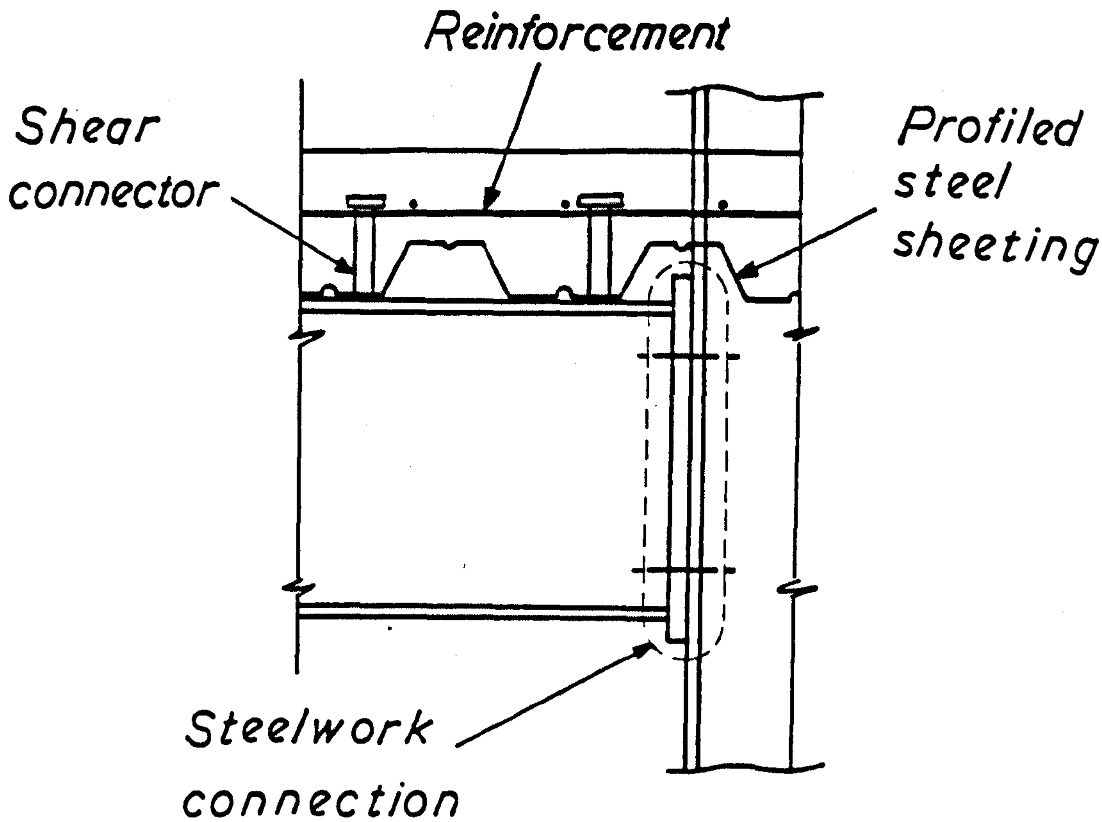
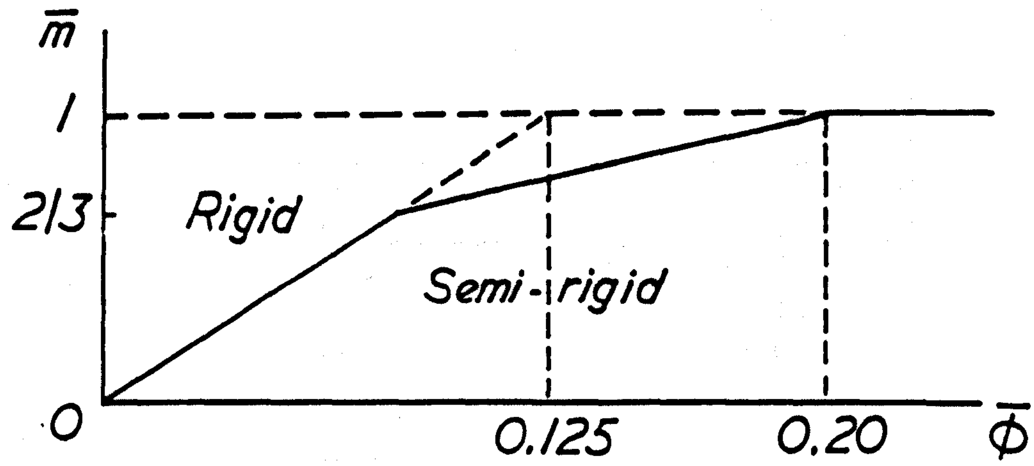


Fig. 4-2, A typical composite beam-to-column connection



$$\bar{m} = \frac{M}{M_{pl.Rd}} \qquad \bar{\Phi} = \frac{E I_b \Phi}{L_b M_{pl.Rd}}$$

Fig. 4-3, Classification of connections in Eurocode 4

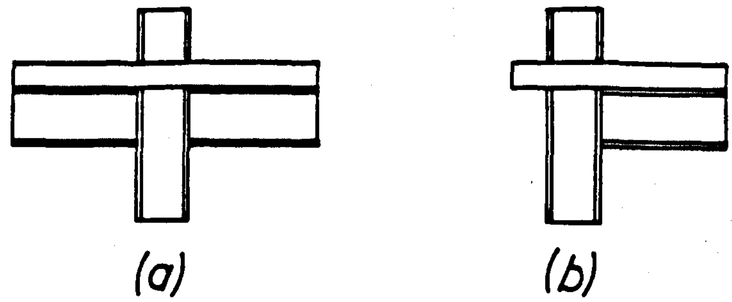


Fig. 4-4, (a)Cruciform specimen,and (b)cantilever specimen

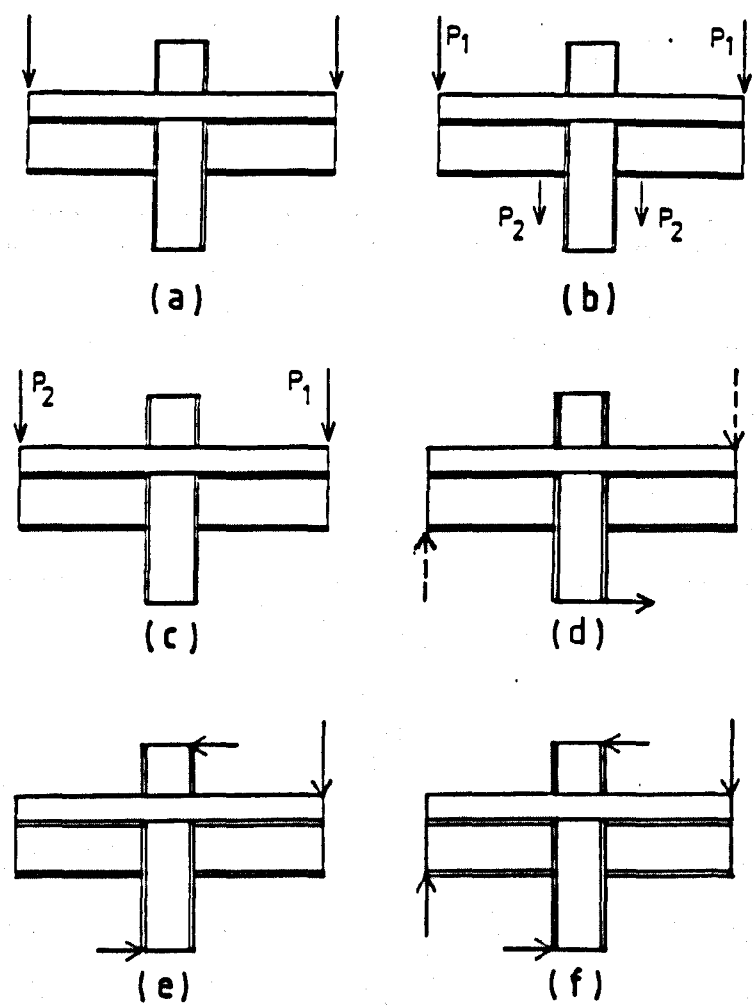


Fig. 4-5, Loading arrangement of tests

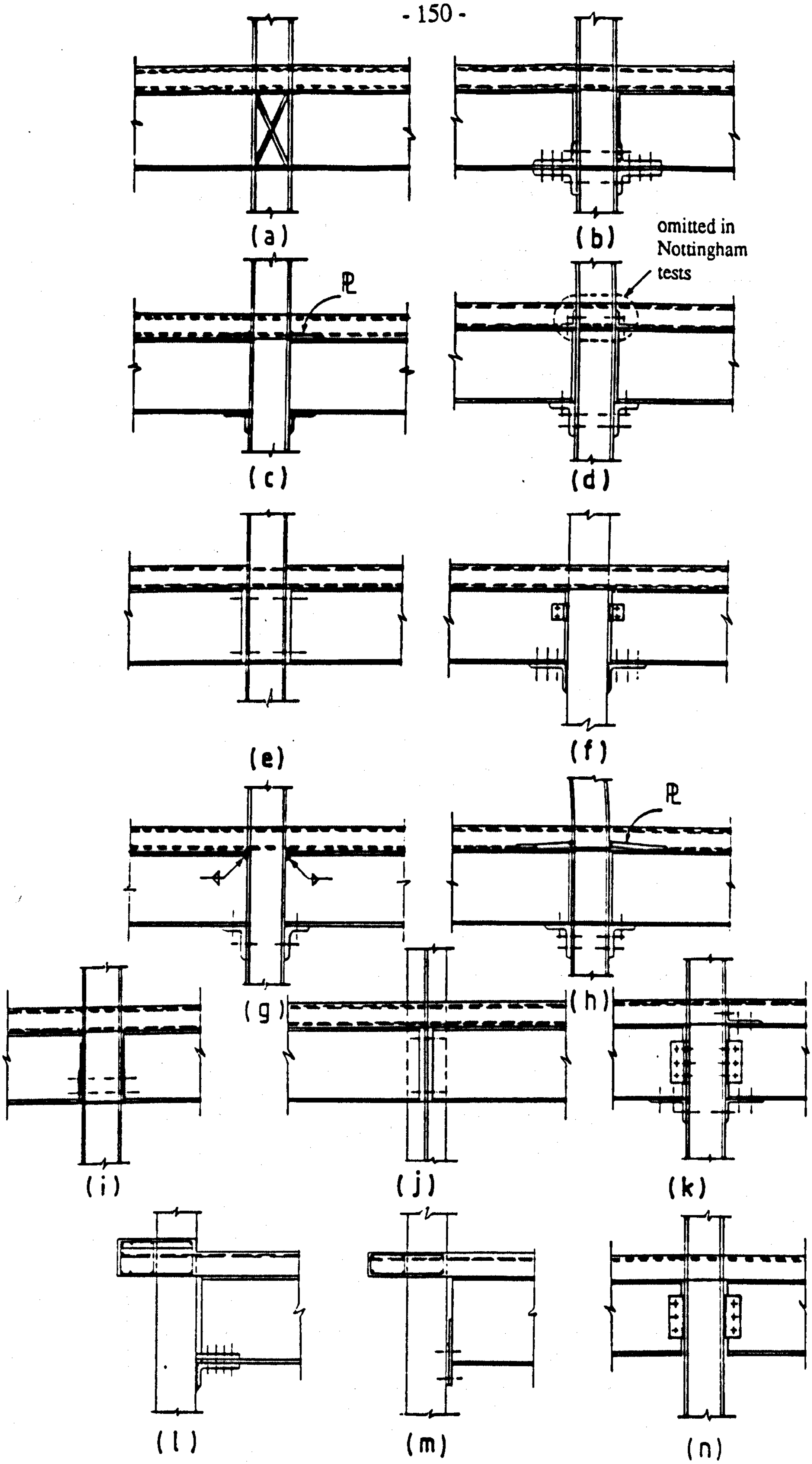


Fig. 4-6, Types of composite connections used in tests

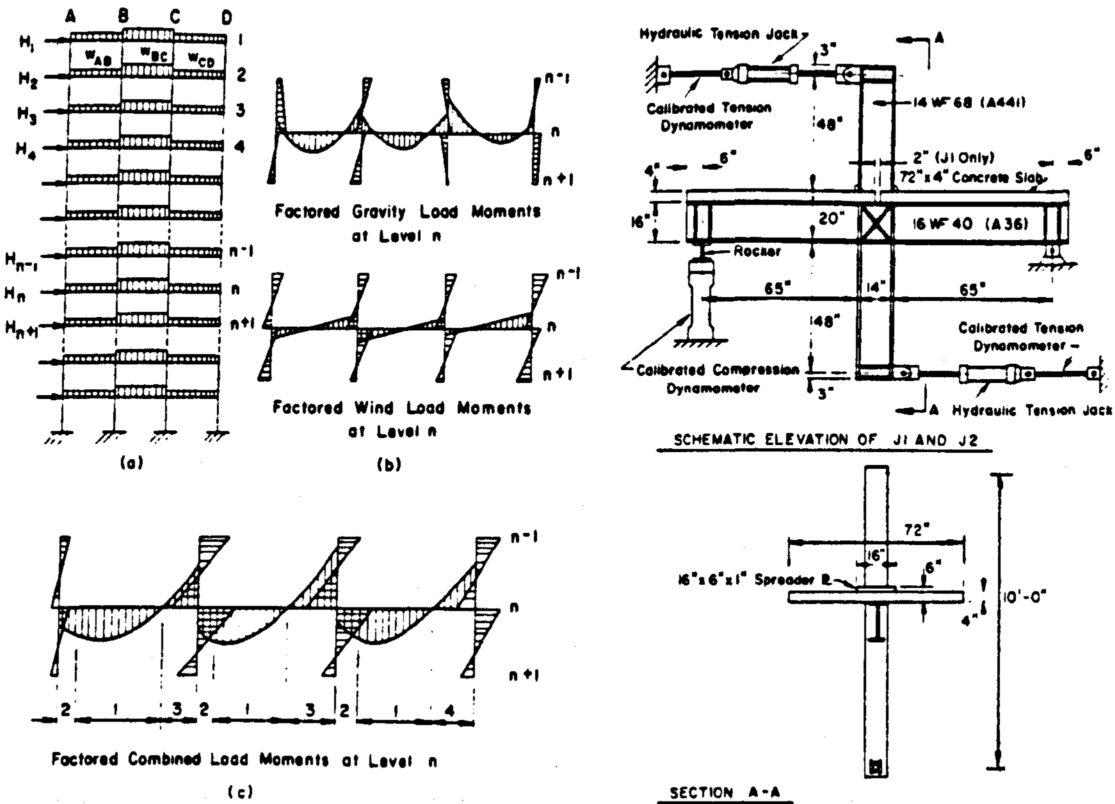


Fig. 4-7, Bending moments in unbraced multistory frames, and the shematic view of test specimens (Daniels et al)

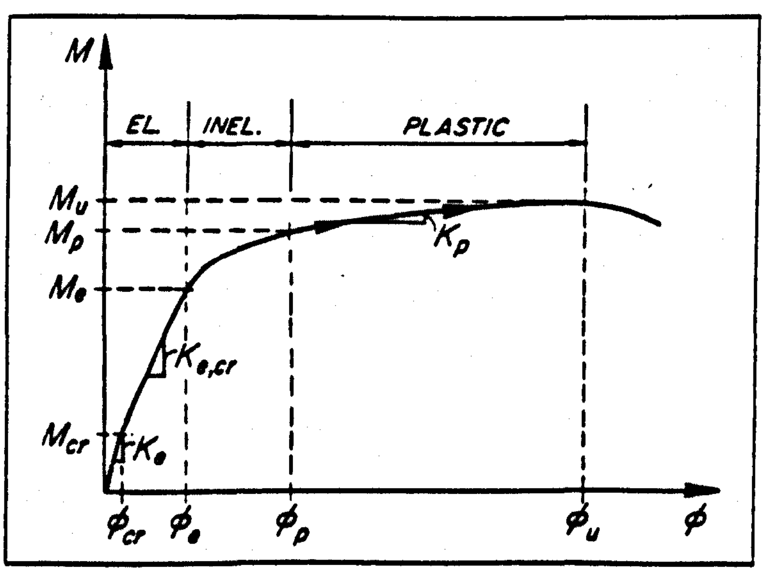


Fig. 4-8, Typical connection behaviour (Benussi et al)

Chapter 5

TESTS ON COMPOSITE CONNECTIONS

5-1-Introduction

In order to achieve economic design, a designer needs to know the behaviour of each individual connection, obtained either by tests on the actual joints or by using prediction equations. It is usually not feasible to arrange tests on the joints to be used in a structure and therefore some rules are needed for calculating their characteristics. A large amount of data is available from previous tests on steel connections, which has been used by several researchers for predictions of connection stiffness. Some methods have recently been employed in the codes of practice, such as Eurocode 3 (1992).

In comparison with steel connections, there is a lack of experimental information necessary for the development of a design criterion for composite connections. This fact has well been reflected in Eurocode 4(1992), as commented on in Chapter 4. The test programme described herein was planned to contribute to the knowledge of composite connections and the derivation of an appropriate prediction equation.

5-1-1-The Selection of Connection Details

A number of tests on the behaviour of steel beam-to-column connections in fire have been recently conducted by Lawson et al(1990). Three types of connections were tested as being typical of those used in practice:

1) "Rigid" connection: extended end plate.

2) "Semi-rigid" connection: flush end plate.

3) "Flexible" connection: web cleats(double sided).

The flush end plate and the web cleat connections were supporting composite slabs. It was decided to base the selection and design of the test specimens on those used in the fire tests. The extended and flush end plate connections were selected with the same size and dimensions. The aim was to concentrate on the end plate joints.

The end plate joints are extensively used in steel construction. They are fabricated neatly and once welding is complete at the workshop, the beam section becomes less vulnerable to distortion caused by handling. The frames utilizing this type of joints are also erected more rapidly and safely. The horizontal slip that occurs in cleated joints is avoided in end plate joints. Vertical slip has rarely been reported in the end plate joints. The High Strength Friction Grip bolts therefore are not required. Steel fabricators prefer the use of ordinary bolts rather than costly HSFG bolts.

In present UK practice, high yield bars are employed for reinforcing concrete. The use of profiled steel sheeting as the formwork for concrete is increasing which also provides a platform for construction work. It was therefore decided to use high strength bars and the steel decking. The same type of reinforcement and sheeting had been used in the fire test. The connection tests being conducted nowadays in the research centres have utilized the same slab thickness and similar type of bars and decking, comparison therefore will be possible between the results of these tests.

There is a lack of data concerning composite connections acting about the minor axis of the column. With reference to Fig. 4-1, a column may support beams from one side or more via the joints to web or/and flange of the column. As described in Chapter 3, the overall stiffness of the column is highly dependent on the restraint provided by the beam-to-column connection. The slenderness of the column limits its load carrying capacity which is controlled by the stiffness about its minor axis. A number of tests were planned to study the restraint provided by minor axis composite connections to the steel column. The steelwork connection and reinforcement were taken similar to those of major axis tests.

A total of eleven tests on end plate beam-to-column connections under static loading were performed. The test programme is summarised in Table 5-1. All tests were carried out on cruciform specimens to model the internal joints of the structures. In seven tests, the end plates were bolted to the column flanges, i.e. the beams acted about the major axis of the column. In the remaining four, the end plates were bolted together through the holes in the column web, i.e. the beams acted about the minor axis of the column. Each group includes a test on a bare steel flush end plate connection with the same details as the composite connections, except for the omission of the concrete slab.

The amount of reinforcement was chosen so as percentages of about 0.5%, 1.0%, and 1.5% could be obtained. The percentage is the ratio of the total area of the main reinforcing bars to the area of concrete above the ribs of profiled steel sheeting.

The variables in the tests were the type of end plate connection, the type of loading, the depth of the steel beam, the amount of reinforcement and the degree of shear connection. The common parameters were as follows:

Structural steelwork(Grade 43)	Beam: 305x165 UB 40*
	Column: 203x203 UC 52
	End-plate: 15 mm thick
Bolts	20 mm diameter Grade 8.8
Deck	PMF CF46 0.9 mm thick
Slab	1100 mm wide, 120 mm deep overall
Concrete	Normal weight, designed as Grade 30
Reinforcement	A142 mesh plus T12 bars
Shear Connectors	19 mm stud x 100 mm long before welding

* Except in Test 10 which was 457x152 UB 52

The steelwork details of major axis tests are shown in Figs. 5-1 and 5-2, including the arrangement of decking and studs. The steelwork details and arrangements of the minor axis tests are shown in Fig. 5-3. The reinforcement details are shown in Figs. 5-4 and 5-5.

5-1-2-The Design of Connections

In the design of the connections, the yield strength of structural steel was taken from the mills certificates. The yield strength of reinforcing bars, mesh and end plate material were based on measured values. The cross sectional properties have been taken as the tabulated figures. The calculations based on these values were carried out on steel connections for both flush end plate and extended end plate according to EC3(1992). The compression zone was found to be the weakest part, hence it was stiffened like the fire tests to avoid this mode of failure. These calculations are given in Appendix A. Design calculations were also performed for the attached beams. Assuming continuous composite beams, two equal spans of 9.00 m and 11.5 m were taken for 305x165UB40 and 457x152UB52 sections respectively. These calculations are given in Appendix B.

The width of 1.1 m for composite slab was deduced from the recommendations in BS5950:Part3.1 and EC4 for the effective breadth of the concrete flange. To compensate for the greater effective breadth of the deeper beam, i.e. 1.4 m, the strength of concrete was increased by 25% by simply changing the water to cement ratio.

The resistance moments of the composite connections were determined from the resistance of the tension zone of the steelwork connection, R_b, and the resistance of the reinforcement, $R_r = f_{yr} A_r$. If the total resistance $(R_b + R_r)$ exceeded that of the lower steel flange, $R_f = BTf_y$, a depth of web adjacent to this flange was assumed to be stressed to yield(see Fig. 5-6). This depth x, was determined by equilibrium and used in the following formula to calculate the resistance moment of the composite connection:

$$M_c = R_r\left(D_r - \frac{T}{2}\right) + R_b\left(D_b - \frac{T}{2}\right) - (R_r + R_b - R_f)\left(\frac{x}{2} + \frac{T}{2}\right) \tag{5.1}$$

In the case of extended end plate connection, the second term in Eqn. (5.1) which takes account of the connection resistance in tension, should account for the bolt row in the extended part of end plate. Therefore the product of the force in the top row of bolts and its distance from the centre of the bottom flange was added.

5-1-3-Test Specimen and Test Rig

Each specimen consisted of two nominally identical cantilevers. Some indication of the variability of nominally identical connections could then be concluded. The length of the cantilevers were 1500 mm with the steel beam 50 mm longer than the concrete slab. A limited overall height for the column, 1200 mm and 2100 mm for major and minor axis tests respectively, was chosen.

Two independent jacks were used to apply loads on each cantilever, at 1400 mm from the face of the column section(the flange in major axis tests and the web in minor axis tests). Each jack acted on the ends of specimens through an arrangement of rollers and knife edge as shown in Plate 1. This arrangement permitted the beams to rotate about the vertical symmetric plane perpendicular to the beams, but did not allow rotation about the plane of the beams' web. The jacks reacted against a supporting rig of "Meccano" members to the laboratory strong floor.

The base of the column was supported on a ball joint seated on a concrete footing. Consequently, the base was held in position but free to rotate. Side bracings were also provided for the safety of test rig in case of sudden failure or any unpredicted circumstances.

Preparation was made in the rig for minor axis tests to resist unbalanced loads. This was done by means of two tie bars, one of which held the column base in place, while the other was attached to the top of the column. The top tie also acted as an additional means to apply the required out of balance by use of a jack. Two arrangements of the ties (a and b in Fig. 5-7) were possible, to enable a decision on which side to unload to be made during testing.

5-1-4-Fabrication of Test Specimens

The columns and the beams were flame cut from the longer lengths supplied. One end of each beam and column were to be connected to an end plate or base plate and were therefore sawn. The widths, thicknesses and depths of all steel sections and end plates were measured in regular intervals and averaged. The tensile tests were performed for mesh and reinforcing bars. The measured values are given in Appendix C.

The plates were drilled together with the column to ease bolting. For each specimen, the column and beams were laid on a strong welding table in a horizontal position, to align the beams and adjust the components. The assembly was then spot welded to allow for dismantling and completing the welds. The beams and columns were assembled again and the bolts were fastened diagonally, e.g. first top right then bottom left and vice versa. A torque of 200 Nm was applied to the bolts by a wrench on which a dial gauge showed the applied torque.

The profiled steel sheeting was cut according to the drawings, care being taken to overlap and cut the sheeting so that the arrangement of decking was symmetric about the column. The decking was put on the beams after cuts had been made to allow the column through the sheeting. The beneficial positions of the studs (see Fig. 5-8) were then marked and the beam top flange ground at these positions for proper penetration of the stud weld. After through-deck welding of the studs, their heights were measured. Then the edge trim for the decking was placed and riveted to the profiled steel sheeting. The reinforcing bars were put on top of the mesh that was supported by clip spacers on the decking, providing 25mm concrete cover to the main bars. The specimen then was ready to be cast.

The concrete mix was designed as Grade C30P to BS5328(1981) using the sieve analysis of aggregates. The mixture consisted of the following:

a) Aggregate to BS882(1983) with nominal maximum size of 20 mm, blend of crushed and uncrushed, air dried by spreading on the laboratory floor.

b) Ordinary Portland cement to BS12(1989).

c) Tap water from the laboratory.

The mixture in ratios by mass was 190 : 350 : 600 : 1200 for water, cement, fine(less than 5 mm) and coarse(greater than 5 mm) aggregates respectively. Trial mixes were made and tested to confirm the design. The measured workability was 50 mm from the slump test.

Concrete was made in the laboratory in six mixes per specimen, out of which were taken 10 cubes for compression tests and 6 cylinders for tension tests. The 150 mm cubes were tested at 7 days, 28 days and on the day of the connection test. The cylinders (300x150 mm diameter) were tested on the same day as the connection test using the indirect method of "Splitting Tensile Strength Test" to BS1881(1983). The concrete properties are given in Appendix C.

The slab was cast with the column being supported on the laboratory floor and the decking supported on a casting rig. The specimen was cured under a wet hessian, together with the samples. Care was taken not to lift the specimen before 12 days. This time period had been determined from the tests on samples of the final trial mix to provide a minimum concrete strength of 30 N/mm^2 before lifting. Furthermore, the 7 days test was performed to check the increase in concrete strength.

The deflection of the ends of cantilevers was measured at the time of first lifting and was found to be in the range of 0.07 to 0.15mm. The specimen was then placed in the test rig with the slab (or the upper flange of the beams in bare steel tests) in a horizontal position. Side restraints and bottom end restraints were provided, as well as some chocks, to hold the arrangement of knife edge and rollers in position.

5-1-5-Instrumentation

The instrumentation used in the tests is shown in Fig. 5-9. It was designed to measure the following:

a) Rotations: Three different methods were used to measure the rotation of the connection:

1) Transducers: An angle section was welded to the top flange of the steel beam perpendicular to the flange. A pair of transducers were based on the centreline of the column 200 mm apart, facing the angle section. These were to measure the horizontal displacements at the top and the bottom of the angle as shown in Fig. 5-10(a). As the connection rotates, the angle section rotates equally. Normally, the top of the angle moves away from the column while its bottom moves towards the column. Therefore, the sum of these two displacements divided by 200 mm gives the connection rotation. In Tests 1-6, the angle was positioned as close as physically possible to the end plate such that the flexural component of the beam rotation was insignificant. A drawback to such arrangement was the reverse curvature of the top flange as shown in Fig. 5-10(b), which could affect the results. The tested specimens were inspected. Although the length of the region affected by local curvature was measured as a maximum of 60 mm, it was found difficult to quantify the error from the unloaded specimens after testing. Nevertheless, the extent of flange deformation suggested a negligible error. However, it was decided to place the angle section further away from this region in the remaining tests. Fig. 5-10(c) shows the displacements measured by a pair of transducers in Test 2, which was that most likely to be affected by local curvature of the top flange.

2) Manual inclinometers: Z-shaped plates with inclinometer points were welded to the top flange of the beam as shown in Fig. 5-10(a). These were used in major axis tests, but not enough space was available in minor axis tests.

Additional inclinometer points were also placed at the end of each cantilever to measure the end rotation of the beams. A reference position on the column was provided to measure the column rotation. This measurement was used to find all rotations relative to the column centreline. In Tests 1-4 manual inclinometers were used on the top of concrete slab near to the column(see Fig. 5-9). These were to give the connection rotation at the uppermost level of the joint. However, their readings were found to be subject to discrepancies. This was due to cracking of concrete that could occur between or under the inclinometer points. In the latter case, the small plates on which the inclinometer was placed for measurement, could tilt, causing error. The rotations measured by manual inclinometers in Test 1, are plotted against moment in Fig. 5-11(a) for comparison.

3) Electronic inclinometers: From Test 5 onwards, three electronic inclinometers were used to measure the rotations directly. An inclinometer was screwed on the web of each cantilever in the tension zone, to avoid any secondary effects resulting from the web distortion in compression zone. The third inclinometer was screwed on the column to measure the column rotation. The rotation of each connection relative to the column centreline then could be found from algebric sum of the readings. This instrument was found to be a more convenient device. It was also more reliable as the manual inclinometer readings were subjected to human error. Its resolution was limited though to 0.02 mrad. The moment-rotation curves of Test 5 using transducers and electronic inclinometers are compared in Fig.5-11(b). As mentioned above, the transducers were pointing the angle section which was placed near the end plate. Therefore they were potentially liable to error caused by the local deformation of top beam flange. Comparison between the results of transducer readings and those of electronic inclinometers shows that the former are

accurate and can confidently be used.

b) Loads: Loads were measured by use of two load cells of 500 kN capacity. The force in the tie bar of minor axis tests was measured by means of a 100 kN load cell. They were calibrated a few times during the test program and found to perform accurately.

c) Strains: The concrete strains were measured in Tests 1-4 by demec points distributed in 150 mm intervals as shown in Fig.5-9. The strain of reinforcing bars were measured in Tests 1-4 by YL-type resistance gauges on the surface of the bars. Compressive strains of the beam flange and web were measured in all tests except Tests 7 and 11, the former by PL-type strain gauges and the latter by PRS-type rosettes.

d) Deflections: Two transducers were used to measure deflections at the end of cantilevers directly under the loading points (at 1400 mm from the column face).

e) Slips: Horizontal slip at the interface of the concrete slab and the steel section was measured by transducers at the ends of slabs, i.e. at 1500 mm from the column face. Interface slip was also measured in Tests 1-4 at 225 mm from the column face but it was found to be influenced by the deformation of profiled steel sheeting, since the transducers were facing a vertical plate attached to the decking. In the description of the tests in this thesis, the measured slip at the steel-concrete interface is meant that slip at the free end of cantilevers. Vertical slip at the connections was checked by the use of two transducers, each being placed as close as possible to the end plate. However, their reading included the deflection of the beam and the tilt of the specimen. A sudden change in their values was considered to be due to vertical slip.

5-1-6-Data Acquisition System

The instruments were wired to a Solartron Data Acquisition System which was connected to a computer. Readings were being taken through up to 50 channels at pre-set time intervals of 10 seconds. The information passed from data logger was being monitored on the computer screen. This data was also being stored on hard disk.

The data could be printed in two printouts with one minute intervals whenever required. This was to get two prints of one stage of loading to be checked against each other. The second of these prints was used in the assessment of tests as the specimen settled under the load after one minute of interval. A continuous mode was also provided to store and print data continuously in situations at which failure was thought to be approaching. The data logging equipment is shown in Plate 1.

5-2-Testing Procedures

The moment-rotation curves of all tests are shown in Figs. 5-12 to 5-22. In Figs. 5-12 to 5-15 (Tests 1-4), the rotations measured by transducers are given. In Figs. 5-16 to 5-22 (Tests 5-11), the rotations measured by electronic inclinometers are used. The moment-rotation curves obtained by other devices in these tests are given in Appendix E. Moments are the applied moments by jacks at the column face.

In all tests, load was applied through two independent jacks. Deflections at the end of cantilevers were kept to the same order as long as balanced loading was applied. The specimens were initially loaded to 20 kN (28 kNm) on each side, in 5 kN increments. They were then unloaded to 5 kN to check the performance of the apparatus. In general, load was subsequently applied up to two-thirds of the calculated ultimate resistance of the connections, in increments of 10 kN. However, in Tests 1 and 2 the loading was to half the calculated resistance. A further exception was that for bare steel connections the increment was 5 kN. The crack widths in the composite tests were measured at this stage, from Test 3 onwards. The crack widths are given in Appendix F. The specimen was then unloaded to 5 kN to monitor its unloading stiffness. The testing procedure from this stage up to failure varied depending on the type of test.

5-2-1-Major Axis Tests

In Tests 1-3, the specimens were reloaded first in increments of 10 kN, reduced to 5 kN as failure approached.

In Test 4, the specimen was reloaded to above 170 kN (238 kNm), at which fracture occurred in a weld to the beam web. The specimen was unloaded completely to permit re-welding, after which the loading was applied once more up to failure (in this case of the composite beam).

In Test 7, the specimen was reloaded to 180 kN (252 kNm) in increments of 10 kN, and then to 205 kN (287 kNm) in increments of 5 kN. The load was further increased to above 210 kN (294 kNm), which was the predicted resistance. Increments of 50 mm in deflection at the end of each beam were made from this stage onwards. The loading continued until a rotation of 55 mrad was achieved in the north connection.

Test 9 was a flush end-plate bare steel connection with a predicted maximum load of 50 kN (70 kNm). The specimen was reloaded to 70 kN (98 kNm). The jacks were then fully unloaded to permit adjustment of the north side loading arrangement. The knife-edge was moved 30 mm towards column (accounted for in later calculations). The previous load level was restored and loading continued until a rotation of 50 mrad was achieved.

In Test 10, the specimen was reloaded to 280 kN (392 kNm), at which a bang was heard. A pause occurred to permit the specimen to be inspected. Vertical slip at the column-end plate interface was found to have occurred. The specimen was unloaded to decide whether any alteration should be made in the connection. The replacement of ordinary bolts with HSFG bolts was considered. The calculation for the resistance of the existing bolts was checked, which confirmed the bolts to be able to carry the maximum predicted load. Therefore the bolts were taken out of their position to see if there had been any deformation. This inspection was done on the bolts one by one, and although no visible change was seen, they were replaced by new Grade 8.8 bolts, as used previously. The specimen then reloaded to 295 kN (412 kNm), at which the reinforcement fractured.

5-2-2-Minor Axis Tests

In these tests, the specimens were reloaded to two-thirds of the ultimate predicted load. Further loading then continued until the onset of failure. This was judged partly from successive measurement of crack widths, partly from the moment-rotation curve and partly from predictions of resistance. Care was taken to avoid fracture of the mesh, which could occur without warning due to its brittleness.

In applying unbalanced loading, it was required to reduce the load on the connection which had reached failure, whilst maintaining constant load on the other connection. Tie bars had therefore to be mounted in the appropriate positions (a) or (b) shown in Fig. 5-7.

In Test 5, the out-of-balance was achieved by applying tensile force to the top tie bar, accompanied by adjustments to the jacks providing vertical load. This procedure continued until a maximum difference of 90 kN in load (125 kNm in moment) was achieved, by which stage plasticity had developed in the column. This coincided with the end of available stroke in the jack acting on the tie bar. This was followed by a reverse procedure to restore balanced loading.

In this test, despite flattening of the curve from a rotation of 10 mrad onwards, the initial balanced load was increased until a rotation of 25 mrad was achieved. This was judged to be the minimum rotation capacity required of the connection. Before unbalanced loading was applied, both jacks were fully unloaded to permit adjustment to the alignment of jacks relative to the rollers beneath, through which load was applied to the slab. The previous level of balanced loading was then restored and only then did the unloading of one side commence. After achieving the maximum out-of-balance, balanced loading was restored at a load level of 180 kN (252 kNm), and then both sides were loaded to failure.

In Test 6, the same procedure of unbalanced loading was undertaken at a load level of 85 kN (119 kNm). This procedure continued until a difference of 70 kN in load (100 kNm in moment) was achieved. The reverse procedure was then applied to restore the previous maximum level of balanced loading. Once this was obtained, both sides were loaded to failure.

In Test 8, despite possible further load resistance from the connection, the increase in load was stopped when the maximum predicted load was reached. The unbalanced loading was carried out in the same manner as above. When the difference in loads equalled 80 kN (110 kNm), the reverse procedure was undertaken. As the loading approached the previous maximum level of balanced load, it became necessary to adjust the position of jacks. Both jacks were fully unloaded to permit adjustment. The previous level of balanced loading was then restored. Loading continued until failure of the specimen.

In Test 11, the specimen reloaded to 60 kN (84 kNm) at which the unbalanced loading applied. The tie bars were mounted and one side of specimen unloaded to 10 kN, while the load on the other side was kept at 60 kN. Balanced loading was then restored and increased to 75 kN (105 kNm). The specimen was then completely unloaded to align the arrangement of load cell and knife edge. It was then loaded to previous level. As the specimen tended not to sustain more load, it was unloaded back to 60 kN, and second procedure of unbalanced loading carried out. This was to compare the unloading stiffness of the connection with that obtained previously, since they corresponded to different levels of plasticity. After one side had been unloaded to 5 kN, it was loaded again. The specimen failed due to the stripping of top bolt row, below 60 kN.

5-3-Test Observations

Plates 2 to 12 show the specimens of Tests 1 to 11 respectively. The results of all tests are summarized in Table 5-2. The moment at failure, M_c, is calculated at the face of the column, including 3 kNm and 1 kNm for the self weight of the slab and steel beam respectively. The rotation ϕ_c corresponds to the maximum experimental moment and is the average between two sides. ϕ_{ult} is the ultimate rotation at failure of the side that failed. In minor axis tests, these are given for the side that was unloaded in the unbalanced phase. The observed behaviour of the connections are described separately for the major and minor axis tests in the following sub-sections.

5-3-1-Major Axis Tests

5-3-1-1-Test 1

Cracking in the slab, running from the toes of the column's flanges, was visible at 20 kN (28 kNm). As higher load was applied, further cracks, running transverse to the steel beams, formed at increasing distance from the faces of the column. At later stages of loading, the tendancy was for existing transverse cracks to widen, rather than for new cracks to form. Some diagonal and longitudinal cracking was also observed.

Within the tension zone of the steelwork connection, significant deformation occurred to the column flange (first observed at 140 kN (196 kNm)) and, to a lesser extent, the flush end plate. Failure occurred due to fracture of reinforcing bars and mesh in one composite beam, over half the width of the slab. The associated crack was immediately outside the column section. At failure, slight deformation associated with local buckling was visible in the lower flange of the beam.

Immediately prior to failure, deformation associated with lateral-distortional buckling had become visible. Further deformation of this kind was prevented by attaching a restraint from the test rig.

Negligible slip occurred at the steel beam-slab interface. Negligible deformation occurred in the column stiffener.

5-3-1-2-Test 2

The development of cracking was similar to that observed in Test 1. Much less deformation occurred in the steelwork connection. The most noticeable deformation was to the column flanges, which were pulled apart relative to one another. Failure occurred by local buckling in the lower flange and in the lower part of the web of the steel beam.

The initial lateral imperfections of the lower flanges had been recorded before loading as shown in Fig. 5-23. Lateral displacement of this flange at the end of each beam was measured as load was applied. Such displacement became visible at 80% of the failure load. Deformation of this kind was prevented by attaching a restraint, before failure occurred.

5-3-1-3-Test 3

Behaviour was similar to Test 1 except that fracture of the reinforcement occurred at a much smaller rotation. First sign of deformation of column flange was observed at 120 kN (168 kNm). The cracking pattern in the slab was more limited than in previous tests, being restricted to regions of the slab near the column. Lateral-distortional deformation was prevented by web stiffeners placed towards the end of each beam.

5-3-1-4-Test 4

Cracks were first visible at 20 kN (28 kNm) running from the toes of the column flanges. As the load increased, further cracks running transverse to the steel beam formed in intervals of the pitch of the sheeting profile (225 mm), see Fig. 5-24. The development of transverse cracking was slower and cracks were more limited in number than in previous tests. The last new crack formed at 130 kN (182 kNm). At later stages of loading the existing cracks widened and extended towards the slab edges. Cracking on the longitudinal centreline of the beam occurred at 100 kN (140 kNm), over the stud nearest to the column at both sides. These cracks did not extend further until 150 kN (210 kNm). From this stage onwards, they extended rapidly toward the far ends of beams while also opening up in width.

Unexpected cracks appeared around the load points at 140 kN (196 kNm). They seemed to be independent from the formation of the other cracks and became more significant after the specimen was unloaded and relaxed overnight. At 140 kN concrete had become detached from the decking, as found by knocking on the underside of the metal decking.

Signs of beam web buckling became visible when the specimen was reloaded to 160 kN (224 kNm). Above 170 kN (238 kNm), the load was difficult to maintain and attempts at its restoration caused the web of one beam to separate noticeably from the end-plate. Further inspection revealed that weld failure had taken place, due to the insufficient penetration of the fillet weld.

The web was rewelded and the specimen reloaded. The cracks around one load point widened very noticeably as the load increased. Lateral buckling became visible before 160 kN (224 kNm) and the beam flange started to buckle. Deformation of the ribs of profiled steel sheeting was also noticeable. Meanwhile, the concrete under the load point separated completely from the rest of the slab, resulting in only 5 studs of the original 7 being effective. Except for the separate region under the load point, uplift of the slab occurred relative to the steel section. The load dropped to 150 kN (210 kNm) and further pumping resulted in loss of resistance in the beam. The test was terminated at this stage for detailed inspection of the failed zone.

5-3-1-5-Test 7

Cracks were first visible at 20 kN (28 kNm) running from the toes of column flange. As the load increased, these extended towards the slab edges. At 60 kN (84 kNm), transverse cracks formed at a distance of about 200 mm from the column face on both cantilevers. Further load increments caused new transverse cracks to form. Some of these were across the complete width of the slab, but some first formed around the longitudinal centreline and then extended towards the slab's edges. The first longitudinal crack (130 mm in length) appeared on the north side, commencing at the position of the first stud from the column.

The specimen was unloaded after the crack widths were measured at 130 kN (182 kNm), and then reloaded to the same load level. At this stage new cracks were found as follows: a transverse crack on the south side near the load point; a transverse crack at 450 mm from the column face; further small longitudinal and diagonal cracks.

As the load was increased further, new cracks formed in the same manner as described above. At 160 kN (224 kNm) some cracks were found around the load point at north side. More increase in load caused the cracks to be distributed successively along and across the slab, as well as some which connected two parallel transverse cracks. The cracks around load points became more noticeable.

At 180 kN (252 kNm), flange buckling was visible. The longitudinal cracks on the centreline had now extended up to 500 mm and 250 mm at the south side and north side respectively. Transverse bending of specimen was also visible as shown in Fig. 5-25. The longitudinal cracks opened up as the load increased, together with two transverse cracks in the vicinity of connection. Web buckling also became visible.

At the maximum load of 213 kN (298 kNm), flange buckling was significant, especially on the north beam. Further pumping of the jacks resulted in greater deformation of the beam and fall-off in the load. The test was terminated when a rotation of 55 mrad in the north connection was reached.

5-3-1-6-Test 9

The beam at the north end rotated in the plane of its cross section at the initial stages of loading. This was considered to be due to the imperfection of steel section around the end of the cantilever, as the clear depth between the toes of flanges each side of the web differed by 3 mm. The load tended to level the top flange.

The specimen was loaded to 35 kN (49 kNm), being two-thirds of the calculated load. At this level, deformation was visible in the column flanges. It was measured as 1.5 mm between the top of the end-plate and the column flange. As the specimen was unloaded gradually to 5 kN, the gaps closed up. Also the deformation of column flange disappeared.

When the load was restored to 30 kN (42 kNm), the column flange started deforming visibly again. At 40 kN (56 kNm), load was found to be dropping during the inspection of specimen. The deformation of the column flange became more apparent as the load reached 45 kN (63 kNm). The gap between the top of end-plate and the column flange was 2.5 mm at this stage. At 55 kN (77 kNm), the gap was 5 mm with significant deformation in the column flange and bending in the top bolt row.

Load was increased in increments as low as 2 kN. As the deformations increased further, it was decided to control the deflections rather than the load; increments of 5 mm were applied. At an end deflection of 35 mm, signs of deformation of end plate above the top bolts were found, corresponding to 63 kN of load (88 kNm of moment). At 55 mm of deflection (70 kN of load, 98 kNm of moment), when the test paused for safety checks, the gap was 12 mm with a rotation of above 34 mrad.

After re-positioning the jacks, the loading was restored to 70 kN (98 kNm). When it reached 73 kN (102 kNm), bending of the top bolts became noticeable and it was thought that one of them was stripping. Loading continued until a deflection of 80 mm was attained, at which the rotations were 50-55 mrad, a maximum load of 75 kN (105 kNm) having been reached. No signs of distress were found in the steel section.

5-3-1-7-Test 10

In Test 10, there was initially a gap between end plate and the column flange, the maximum being 0.8 mm in mid-distance between the two bolt rows. First cracks formed at 50 kN (70 kNm) which was much higher than previous tests. These cracks started from the toes of column flanges. As load increased, some small cracks formed in the vicinity of connection. At 80 kN (112 kNm), previous cracks from the toes of column flange extended towards the edges of slab. At 110 kN (154 kNm) and 130 kN (182 kNm), complete transverse cracks formed on both sides. They were exactly on the first bar of the mesh reinforcement.

It was at 170 kN (238 kNm) that the first longitudinal crack formed on the centreline of north beam near the column. The crack widths were measured at this stage. The initial gap between the end plate and column flange was still unchanged.

During the restoration of load from 5 kN to 170 kN, some short cracks formed together with a new transverse crack over the half slab width on the overlap of the decking. The column flange appeared to start deforming at 180 kN (252 kNm). At 200 kN (280 kNm) and 220 kN (308 kNm), complete transverse cracks formed on the second and third bars of the mesh reinforcement on both sides. Cracks also occurred on the first longitudinal rebars both sides of the column.

At 245 kN (343 kNm) noticeable deformation of the column flange could be seen. As load increased to 250 kN (350 kNm), previous cracks extended, particularly those on the rebars. Two complete transverse cracks at a distance of 1100 mm from the column face also formed on both sides.

As more load was applied, the deformation of column flange became significant. Load could still be sustained until it reached 280 kN (392 kNm). Continuous pumping of the jacks increased load to 284 kN (398 kNm) at which vertical slip occurred at the connection interface. This slip was recorded as 4 mm. Load dropped to 189 kN (265 kNm). This slippage caused cracks in the vicinity of the connection, particularly in the zone between the web and flanges of the column.

As the specimen was unloaded to inspect the bolts, the deformations reversed in the main. During reloading, at 210 kN (294 kNm), some cracks formed around the load points. Cracks opened generally, especially the longitudinal ones on the centreline of the beams. Load was restored to the maximum that reached previously (284 kN). Some diagonal cracks formed at this load level.

The specimen had difficulty in sustaining more load. At 295 kN (412 kNm) the mesh fractured and the load dropped to 275 kN (385 kNm). Continuous pumping resulted in the fracture of main rebars across half width of the slab. Load dropped to 140 kN (196 kNm) and the specimen was then unloaded completely.

5-3-2-Minor Axis Tests

5-3-2-1-Test 5

Due to the stiffness of the minor axis connections, the first cracking only occurred after a load level of 40 kN (56 kNm) was reached. A complete transverse crack ran from the column flange face and extended across the width of slab. The next cracking (at 60 kN) occurred at different distances from the column, depending on the side being examined (see Fig. 5-26(a)). Both cracks at 60 kN (84 kNm) appeared to be on the outer edge of the profile rib (Fig. 5-24). The development at higher load levels comprised mainly transverse cracks, although some diagonal cracks ran from one transverse crack to another. At 170 kN (238 kNm) short cracks formed connecting the toes of column flanges and running across the first studs. There was no longitudinal cracking over the studs, although a few were observed over rebars.

During unbalanced loading, the initial cracks opened up, particularly those over the first studs.

Following the restoration of balanced loading, failure occurred due to fracture of reinforcing bars and mesh near the transverse centreline (Fig. 5-26(a)) over half the width of the slab. This took place immediately after the loading had been re-balanced at 180 kN (252 kNm). This load was significantly lower than the maximum of 206 kN (288 kNm) achieved in the balanced phase of the experiment.

During unbalanced loading, bending of column had been visible before its calculated minor axis elastic resistance moment was reached. At the maximum out-of-balance, the calculated plastic resistance moment of the column had been attained, causing excessive deformation in the column in the vicinity of the concrete slab. Bearing against the slab caused crushing of the concrete in front of the toes of the column flanges.

Due to the configuration of the connections, deformation of the end-plates could not be properly seen, nor the column web. Nevertheless, signs of distress around the top row of the bolts became visible at 170 kN (238 kNm). The most noticeable deformation was the local buckling of the beam's web which began at 140 kN. It increased later and resulted in the deformation of beam's flanges at 170 kN.

After failure, load dropped to 136 kN (190 kNm) on both sides and the test was terminated. Negligible slip occurred at the steel beam-slab interface. No visible deformation was found in the column when inspected after dismantling, except that in the column flanges caused by moment about minor axis in the unbalanced phase.

5-3-2-2-Test 6

The formation of cracks is shown in Fig. 5-26(b). The first transverse cracks formed close to the column at 25 kN (35 kNm) on the first mesh bar over half the breadth of the slab. The crack over the other half was on the mesh at the transverse centreline and coincided with the edge of the rib of decking.

At 70 kN (98 kNm), a complete transverse crack formed at a distance of 1000 mm from transverse centreline on the south side. No new cracks appeared in the unbalanced phase except for some small ones near the column. Limited longitudinal cracks formed from the toes of column flanges and on the mesh bars before and after the unbalanced phase. Comparison with Fig. 5-26(a) for Test 5 (with 1% reinforcement) shows that very limited cracking formed in Test 6.

Crack widths were measured at 35 kN (49 kNm), 55 kN (77 kNm) and 83 kN (116 kNm). This measurement was used to assist in deciding on the stage at which unbalanced loading should start.

After balanced loading had later been restored, a bar of the mesh fractured, with load dropping from 97 kN (136 kNm) to 86 kN (120 kNm). Pumping the jacks continued, causing another bar to fracture and the load to drop to 83 kN. When the remaining bars fractured, load dropped to 49 kN (69 kNm). Further deformation of the specimen resulted in failure in the top row of the bolts and a drop of load to 28 kN (39 kNm). Inspection of the specimen after removal of concrete showed that the top bolts had bent and the thread had stripped in the nuts. Bending of the bolts had been found in Test 5, but not stripping.

The column web was examined, especially around the bolt holes. No signs of distress was found in the vicinity of connection. The diameters of the holes were also measured and no change was found. Nevertheless, the end-plate was deformed and large amounts of strain were apparent around the bolt positions (see Plate 7). Negligible slip occurred at the interface of steel beam and the slab, as measured at the end of slab.

5-3-2-3-Test 8

The development of cracking is shown in Fig. 5-26(c). The first crack was visible on both sides at 30 kN (42 kNm) on the transverse centreline. At 40 kN (56 kNm), short cracks appeared around the column flanges. At 50 kN (70 kNm), the cracks at the toes of column flange extended towards the slab edges. As load increased, complete transverse cracks formed at both sides on the ribs of profiled steel sheeting. At higher loads before the unbalanced phase, new complete transverse cracks formed both sides, the last ones were at 120 kN (S) and 140 kN (N).

Some longitudinal cracks of limited length occurred on the first rebars close to the column, together with some small ones on the second rebars. These cracks extended during the unbalanced loading and as the specimen was loaded to failure.

Failure initially occurred due to fracture in the mesh, and load dropped from 148 kN (207 kNm) to 140 kN (196 kNm). At this stage, signs of local buckling in the beam flange became visible. More load was applied, causing more deformation of the beam flange and eventually the fracture of some rebars. After this failure, load dropped to 123 kN (172 kNm), but was then increased to 130 kN (182 kNm) by continuous pumping. The test was terminated at this stage.

Negligible slip occurred at the end of the cantilevers. Signs of straining around the bolt positions were found on the end-plate, but to a less extent than in Test 6.

5-3-2-4-Test 11

This test was performed to investigate the behaviour of the bare steel minor axis connection which formed the steelwork connection in the composite tests. There was difficulty in observing the deformation of the steel components in the composite minor axis tests, particularly in unbalanced phase. With the concrete slab omitted, it was hoped that these deformations would be visible.

An initial imperfection existed in the end plates as a result of welding. The gaps at the top of the end plates between the column web and the middle of end plate were initially measured as 0.35 mm and 1.25 mm for south and north connections respectively. The gaps were frequently measured during the test.

The specimen was loaded to 42 kN (59 kNm), being two-thirds of the calculated resistance. At this load level, the gaps between the end plates and the column web were measured as 1.5 mm (S) and 2.3 mm (N). As the specimen was unloaded gradually to 5 kN, the gaps closed to the initial values.

When the load was restored to 42 kN, there was still no sign of distress on the end plate around the top bolts. As load increased to 50 kN (70 kNm), the first signs of strain appeared on the end plate and the load found to be dropping slightly during the inspection of specimen.

As load reached 60 kN (84 kNm), which was higher than the calculated resistance, unbalanced loading was commenced. The gaps were 2.6 mm (S) and 3.8 mm (N) at this stage, but from now on their values were influenced by bending of the column.

After balanced loading was restored and more load applied, the specimen sustained the load until it reached 75 kN (105 kNm). Load then began to drop rapidly as further deformation of the end plate was observed. The second phase of unbalanced loading commenced after fall of load to 60 kN (84 kNm).

The specimen was later reloaded to restore the balanced loading, but as this was approached, the bolts stripped in the nuts and the test was stopped for safety. There was no device mounted on the bolts to measure their change in length. Viewing inside the gaps from the top of the connections showed some extension in the bolts. The projected length of the bottom bolts could be compared with those of top bolts showing a difference of 5 mm. Maximum rotation was in the order of 22 mrad.

5-4-Material Tests on Steel Sections and Reinforcement

The tensile tests were carried out on the samples taken from the steel sections after the experiments. The elongation tests were performed for mesh and reinforcing bars. The material test results are given in Appendix C.

5-4-1-Steel Sections

The steel samples were prepared according to BS4360(1986) and tested to BS18(1987).

The samples were taken from the beams used in the connection tests, from zones not affected by heat, buckling or yielding. A length of each beam was first sawn. Two coupons were machine cut from the bottom flange and two coupons from the web. The top flange had been affected by the stud welding, therefore no sample was taken from the top flange. However, the bottom flange of the beam had been in compression and subjected to local buckling. Hence its strength was needed for calculation of buckling resistance.

The samples of the steel column sections were taken from the remaining sections in the batch. Two coupons were machine cut from each column flange and two from the web.

The position and dimensions of the coupon samples are given in Fig. 5-27.

5-4-2-Reinforcement

Further tests were carried out on reinforcing bars and mesh in addition to the initial tests for their yield strength.

The samples of T12 bars were prepared according to BS4360(1986) and tested to BS18(1987). Their standard length was first calculated and the samples were cut from the bars in the batch. The elongation test was performed for four T12 bars. The results of extension at fracture were around 17%.

Four samples were taken from the A142 mesh bars and prepared according to BS4545(1970). The elongation test was carried out on these samples. They appeared to behave in a very brittle manner, their elongation being of the order of 2%.

Table 5-1, Summary of the test programme

TEST NO.	COLUMN AXIS	REINFORCEMENT PERCENTAGE	SHEAR CONNECTION	LOADING TYPE
1	MAJOR	1%	FULL	SYMMETRIC
2	MAJOR	1%	FULL	SYMMETRIC
3	MAJOR	0.5%	FULL	SYMMETRIC
4	MAJOR	1.5%	PARTIAL	SYMMETRIC
5	MINOR	1%	FULL	ASYMMETRIC
6	MINOR	NOMINAL	FULL	ASYMMETRIC
7	MAJOR	1.5%	PARTIAL	SYMMETRIC
8	MINOR	0.5%	FULL	ASYMMETRIC
9	MAJOR	-	-	SYMMETRIC
10	MAJOR	1%	FULL	SYMMETRIC
11	MINOR	-	-	ASYMMETRIC

* The reinforcement percentage is the ratio of the area of main reinforcement to the area of concrete above the ribs of decking.

** Symmetric loading refers to the balanced loading from the beginning to the end of test. Asymmetric loading means that first balanced loading was applied to the specimen, then one side was unloaded and reloaded back to the situation prior to unbalanced loading, which was followed by balanced loading to failure.

Table 5-2, Summary of the test results

TEST NO.	COLUMN AXIS	REIN-FORCEMENT	M_c (kNm)	ϕ_c (m rad)	ϕ_{ult} (m rad)	MODE OF FAILURE
1	MAJOR	1%	262	28.0	35.8	FRACTURE OF MESH PLUS REBARS
2	MAJOR	1%	291	20.0	40.0	LOCAL BUCKLING OF BOTTOM BEAM FLANGE
3	MAJOR	0.5%	179	15.7	26.6	FRACTURE OF MESH PLUS REBARS
4	MAJOR	1.5%	243	9.2	-	SHEAR CONNECTION
5	MINOR	1%	293	22.5*	22.5*	FRACTURE OF MESH PLUS REBARS
6	MINOR	NOMINAL	138	2.8*	11.0*	FRACTURE OF MESH
7	MAJOR	1.5%	302	22.7	55.7	LOCAL BUCKLING OF BOTTOM BEAM FLANGE
8	MINOR	0.5%	207	4.8*	14.2*	FRACTURE OF MESH PLUS REBARS
9	MAJOR	-	105	55.8+	55.8+	DEFORMATION OF COLUMN FLANGE
10	MAJOR	1%	416	14.0	14.0	FRACTURE OF MESH PLUS REBARS
11	MINOR	-	105	22.0	22.0	DEFORMATION OF END PLATE AND BOLT STRIPPING

* These values were measured on the side with lower load in the unbalanced phase.

\+ In Test 9 (bare steel) the load on the specimen could still increase but test stopped due to excessive deformation of column flange.

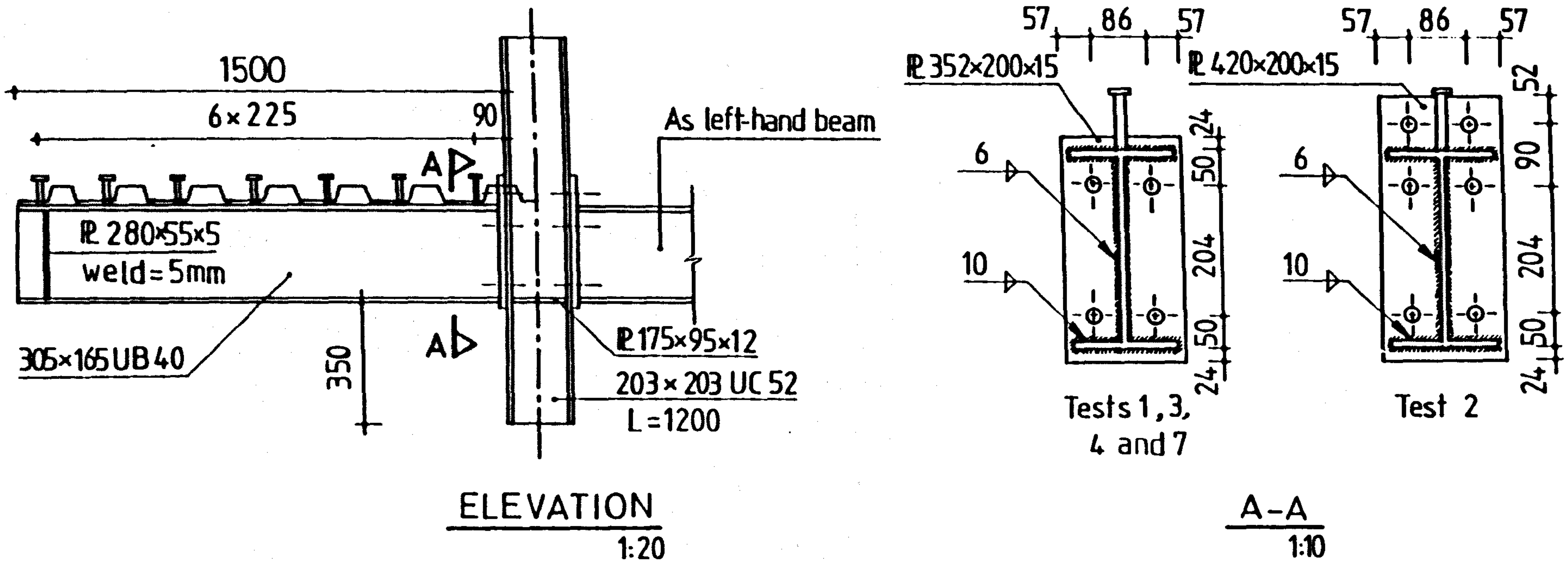


Fig. 5-1, Arrangement of major axis tests (Tests 1,2,3,4 and 7)

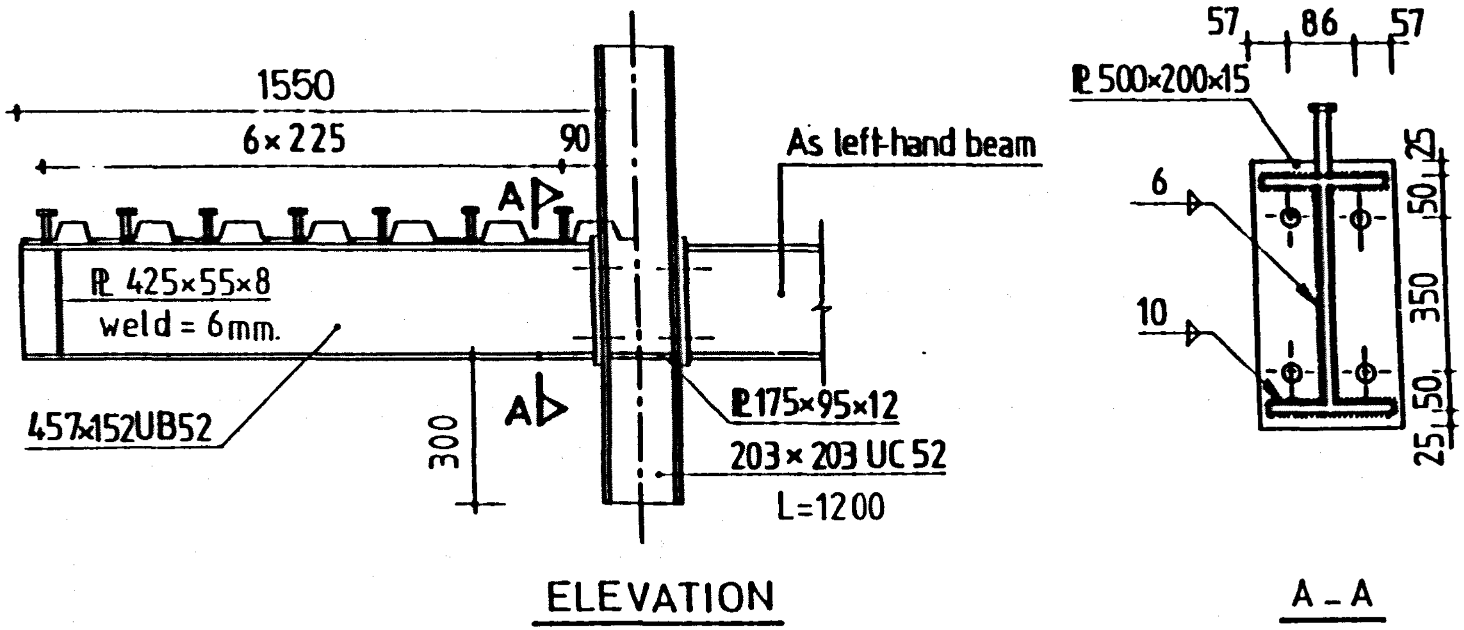


Fig. 5-2, Arrangement of major axis tests (Test 10)

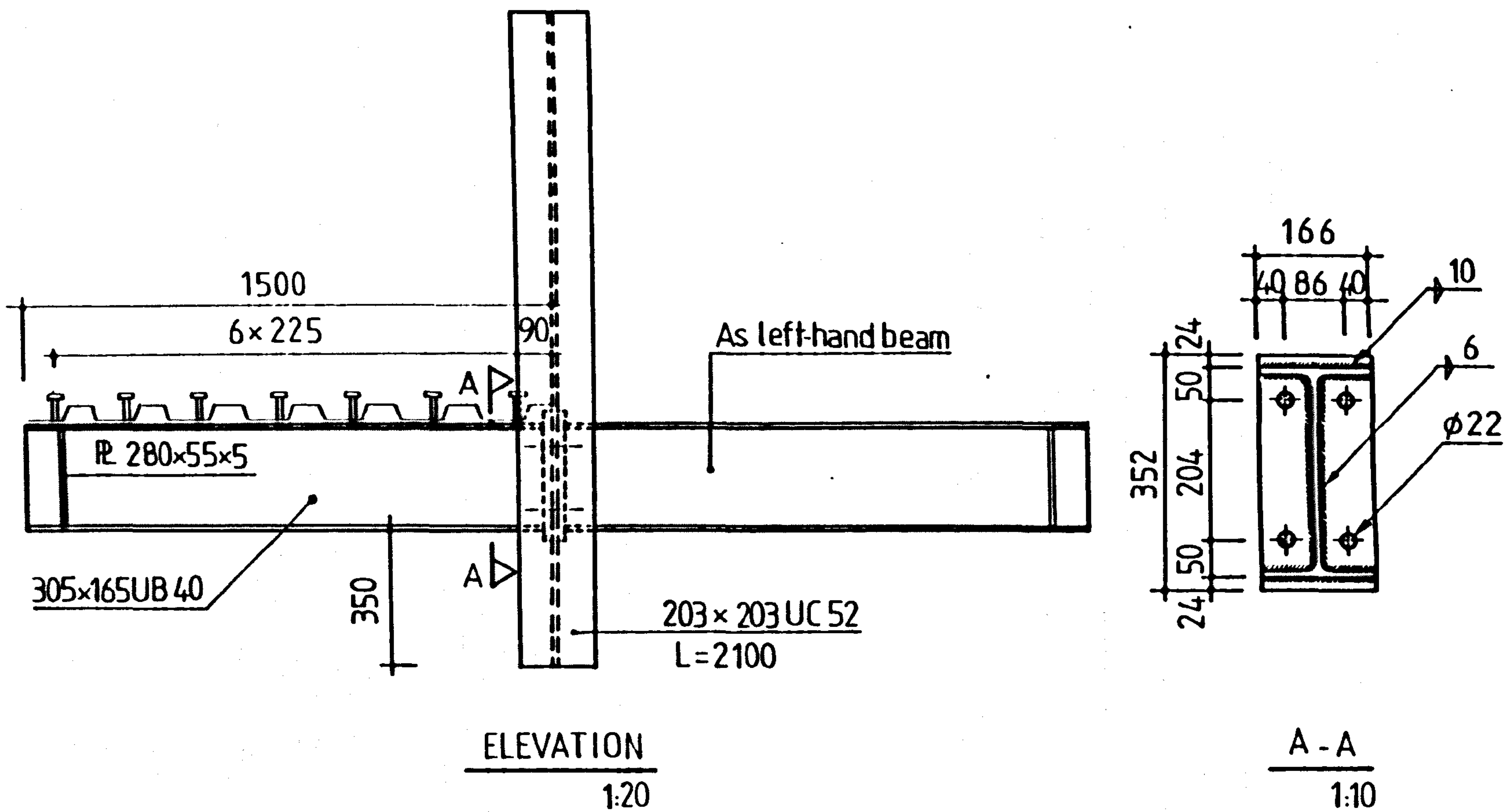
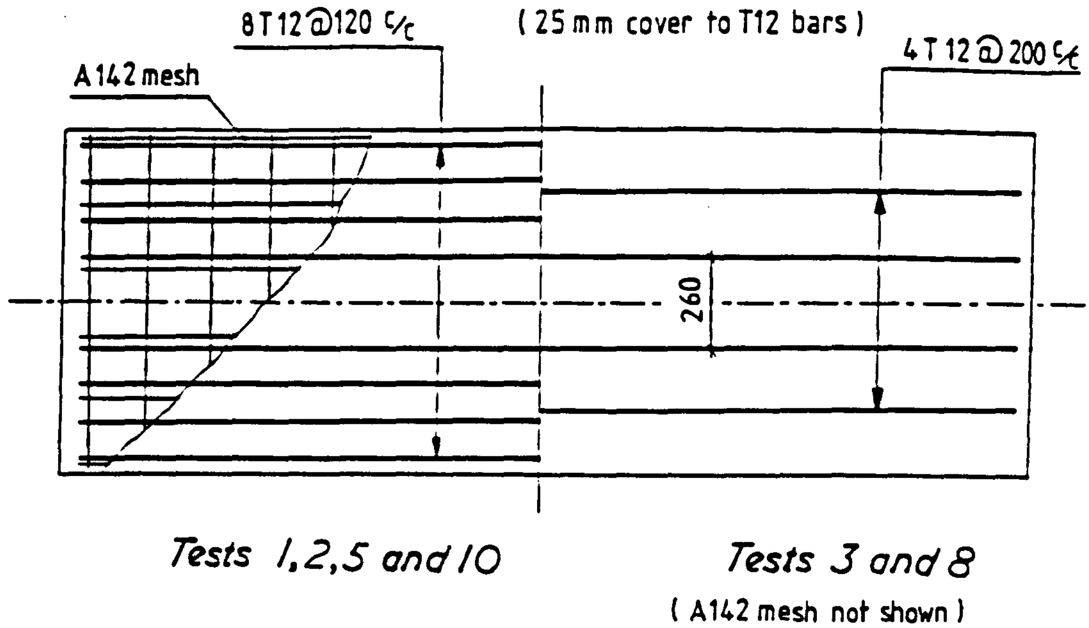


Fig. 5-3, Arrangement of minor axis tests (Tests 5,6 and 8)



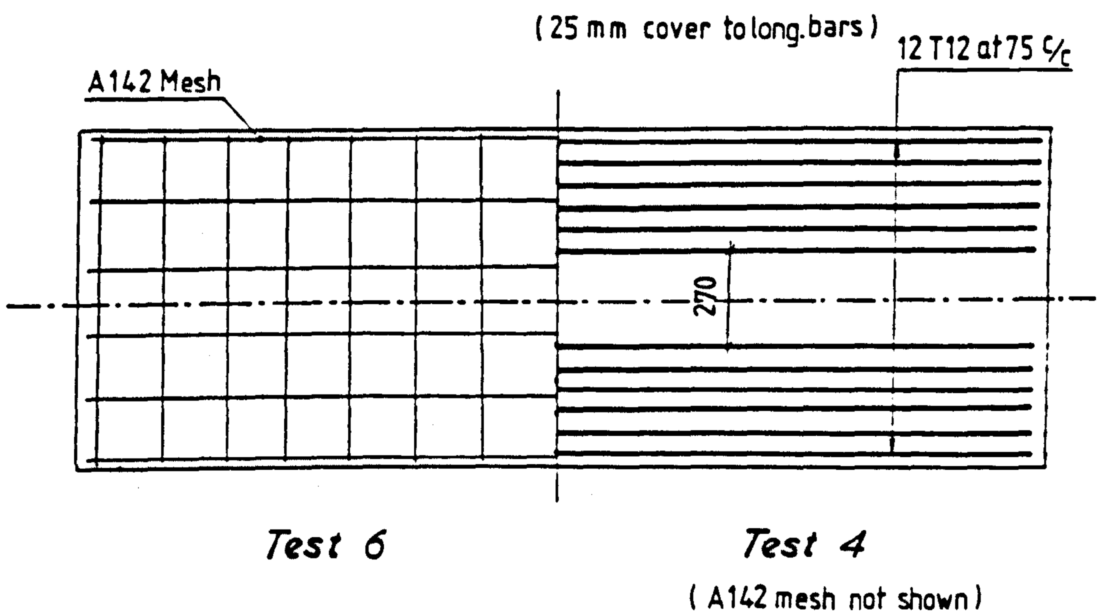


Fig. 5-4, Reinforcement details

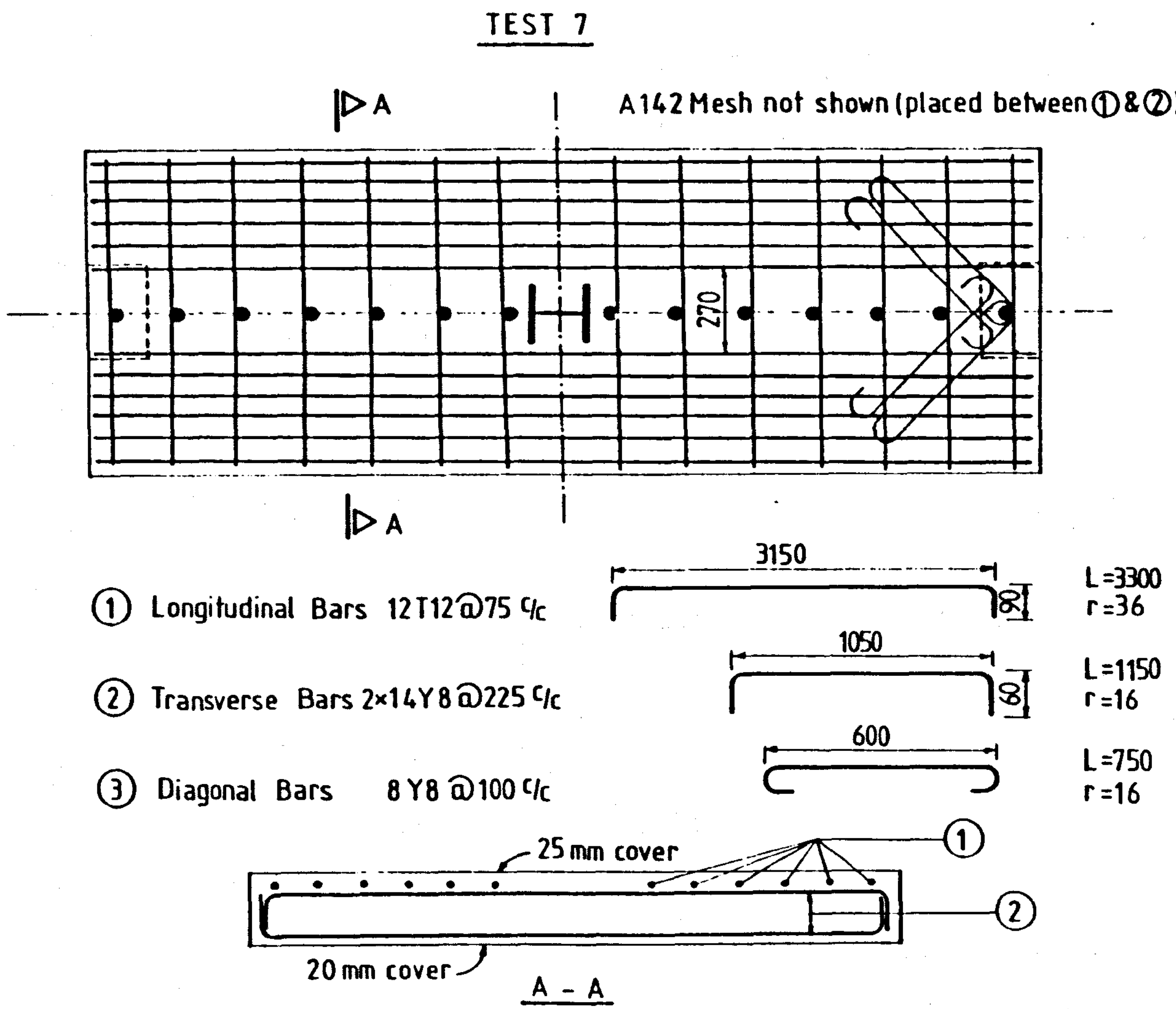
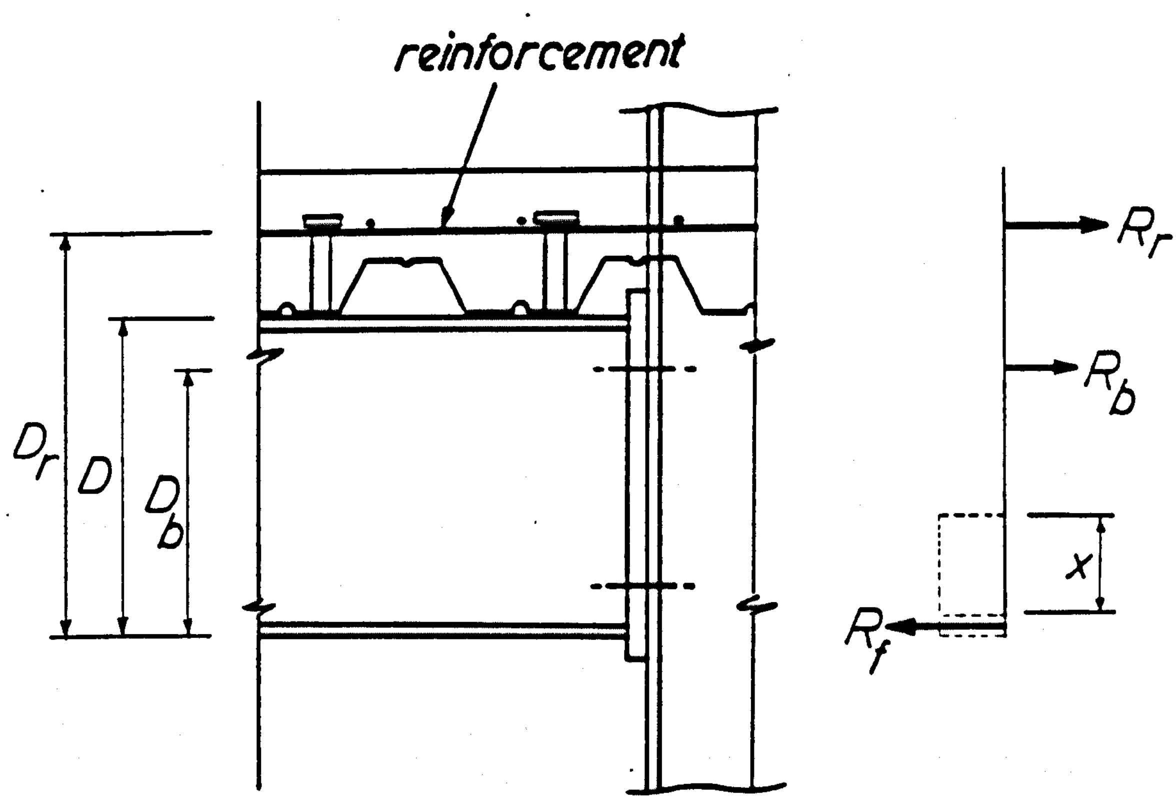


Fig. 5-5, Reinforcement details of Test 7



$$R_f < R_r + R_b \qquad\qquad R_f \geqslant R_r + R_b$$

$$x = \frac{R_r + R_b - R_f}{t \cdot p_y} \qquad\qquad x = 0$$

Fig. 5-6, Notation used in calculation of resistance moment

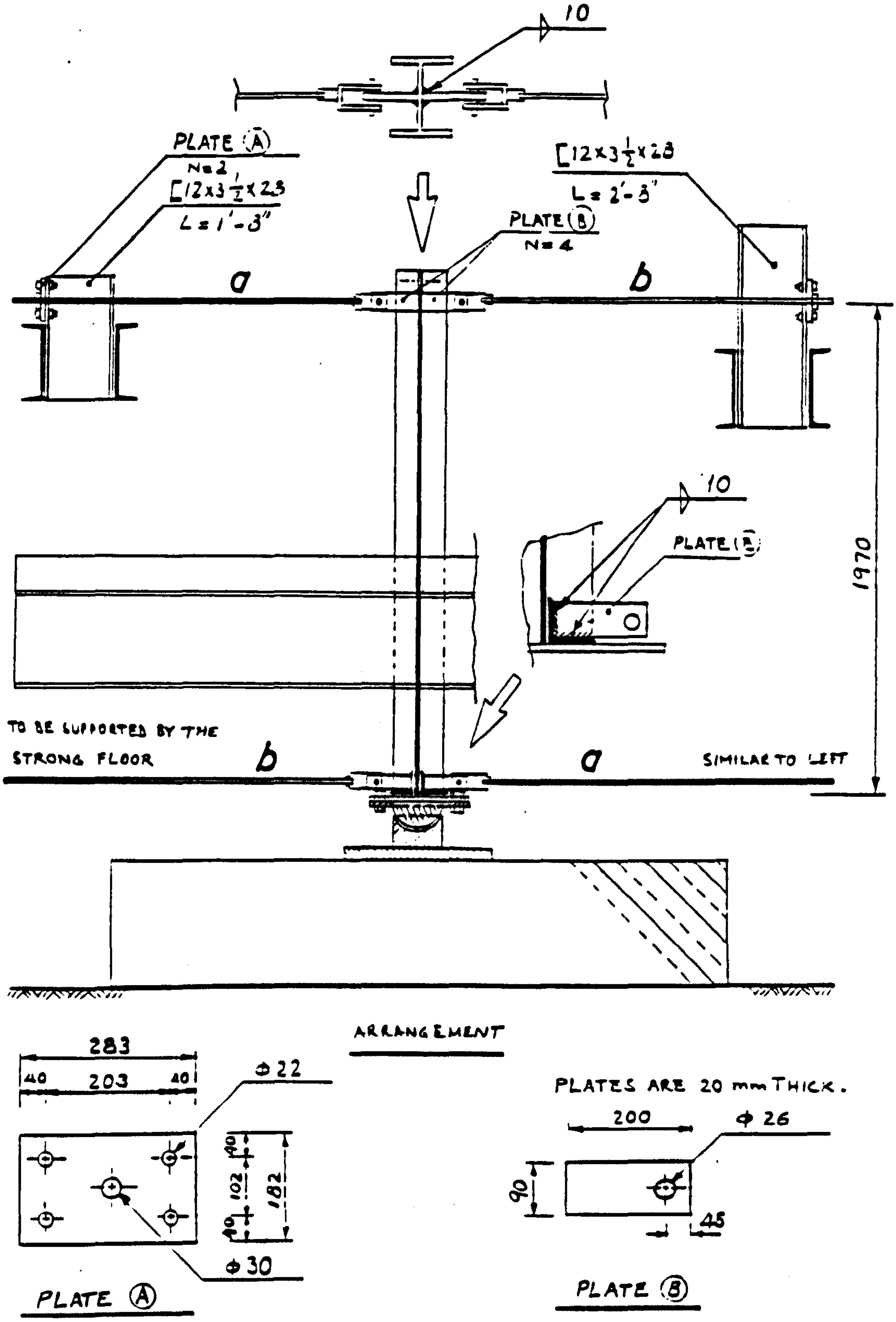


Fig. 5-7, Arrangement of tie bars, either (a) or (b) was used

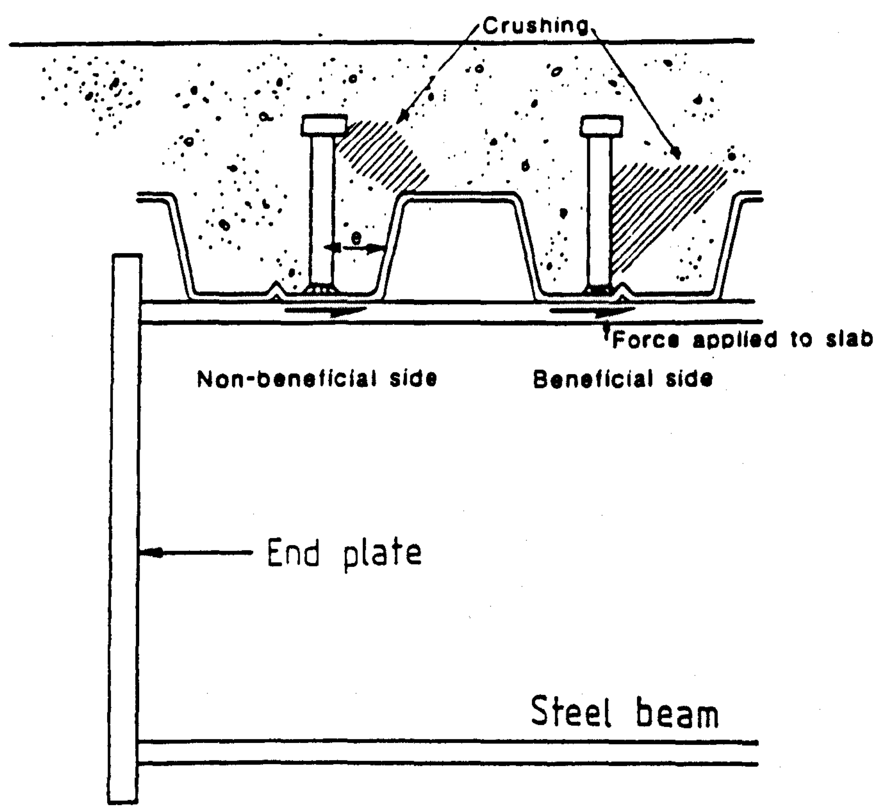


Fig. 5-8, Beneficial position of studs

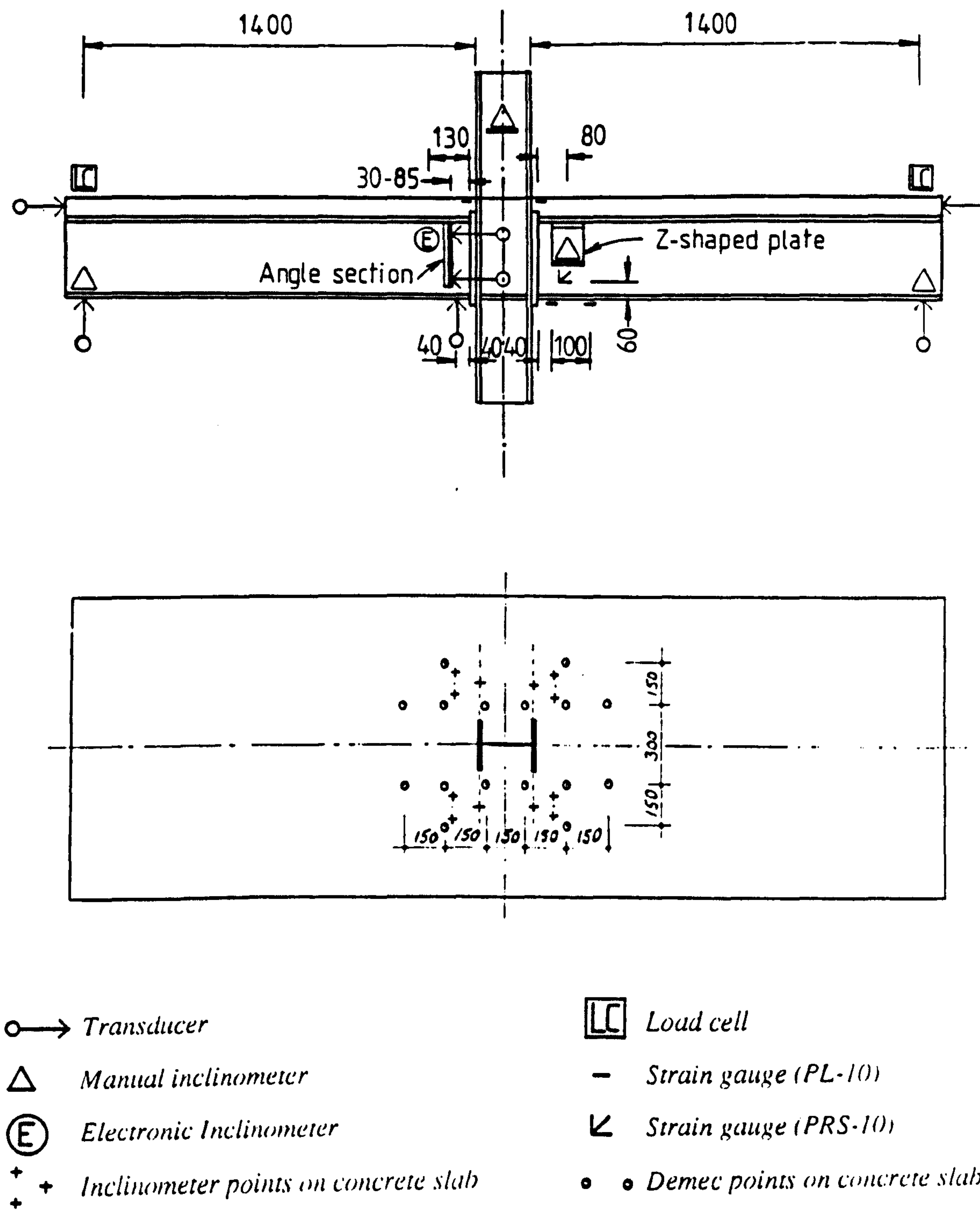


Fig. 5-9, General instrumentation
(Note that all elements of instrumentation were symmetric about column axis)

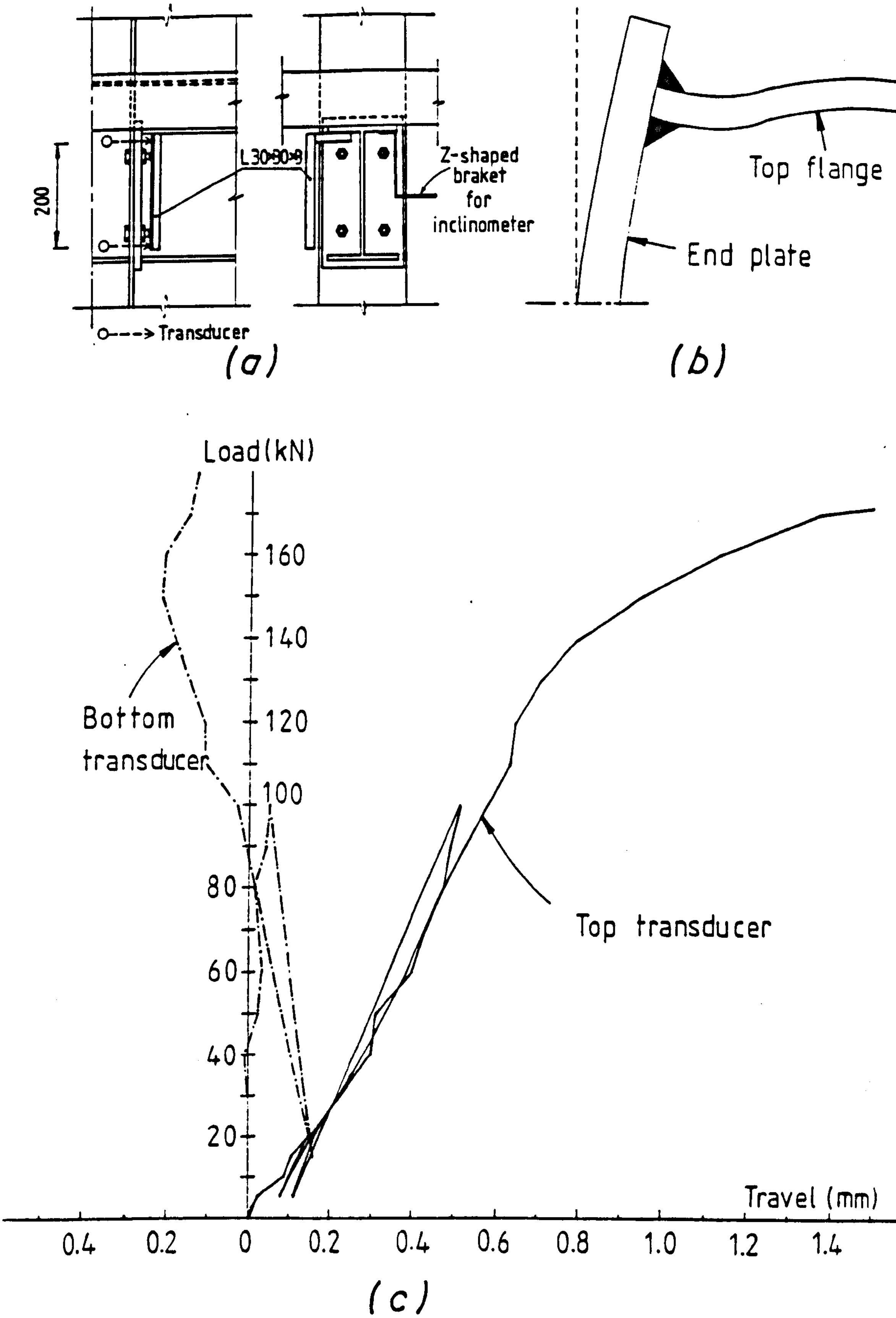
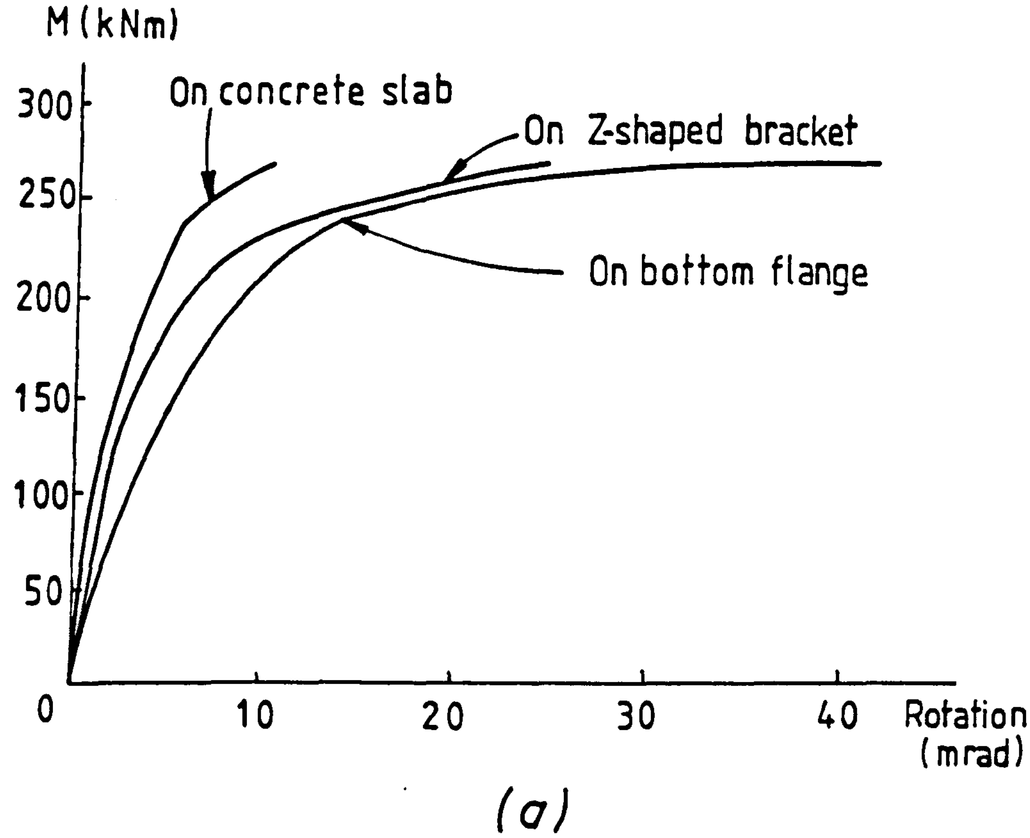


Fig. 5-10, Rotation measurment by transducers



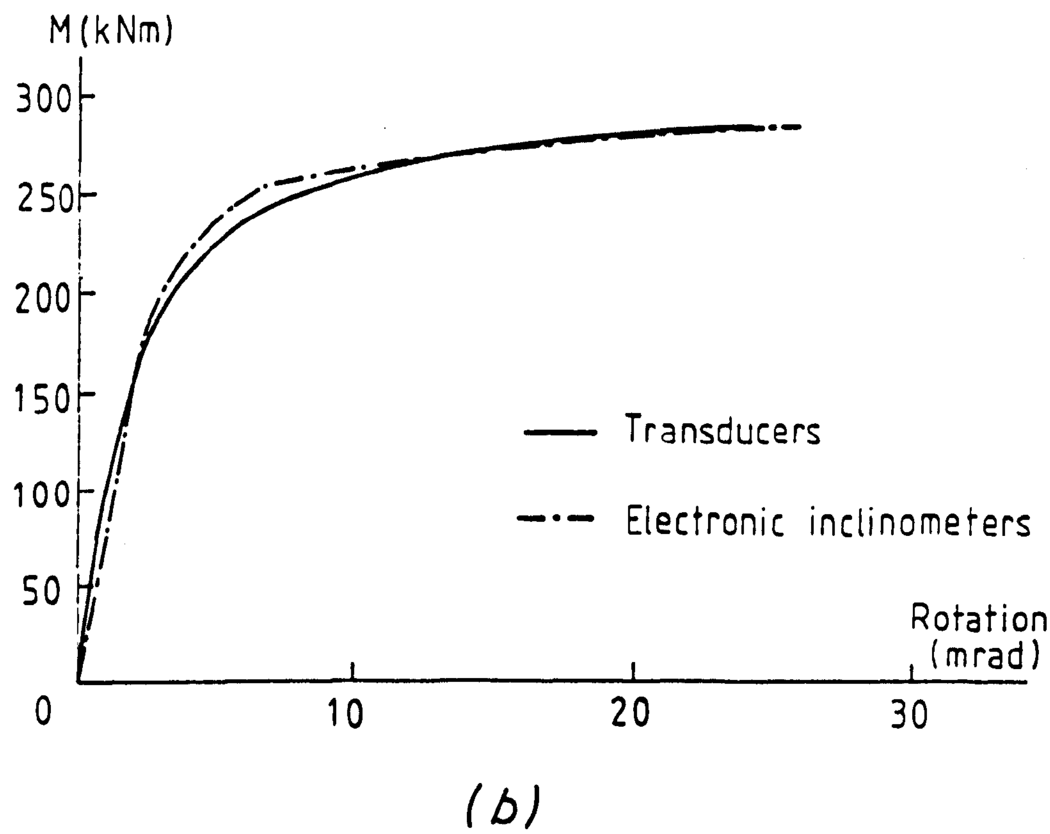


Fig. 5-11, Moment-rotation curves obtained by:
(a) manual inclinometers, Test 1
(b) transducers and electronic inclinometers, Test 5

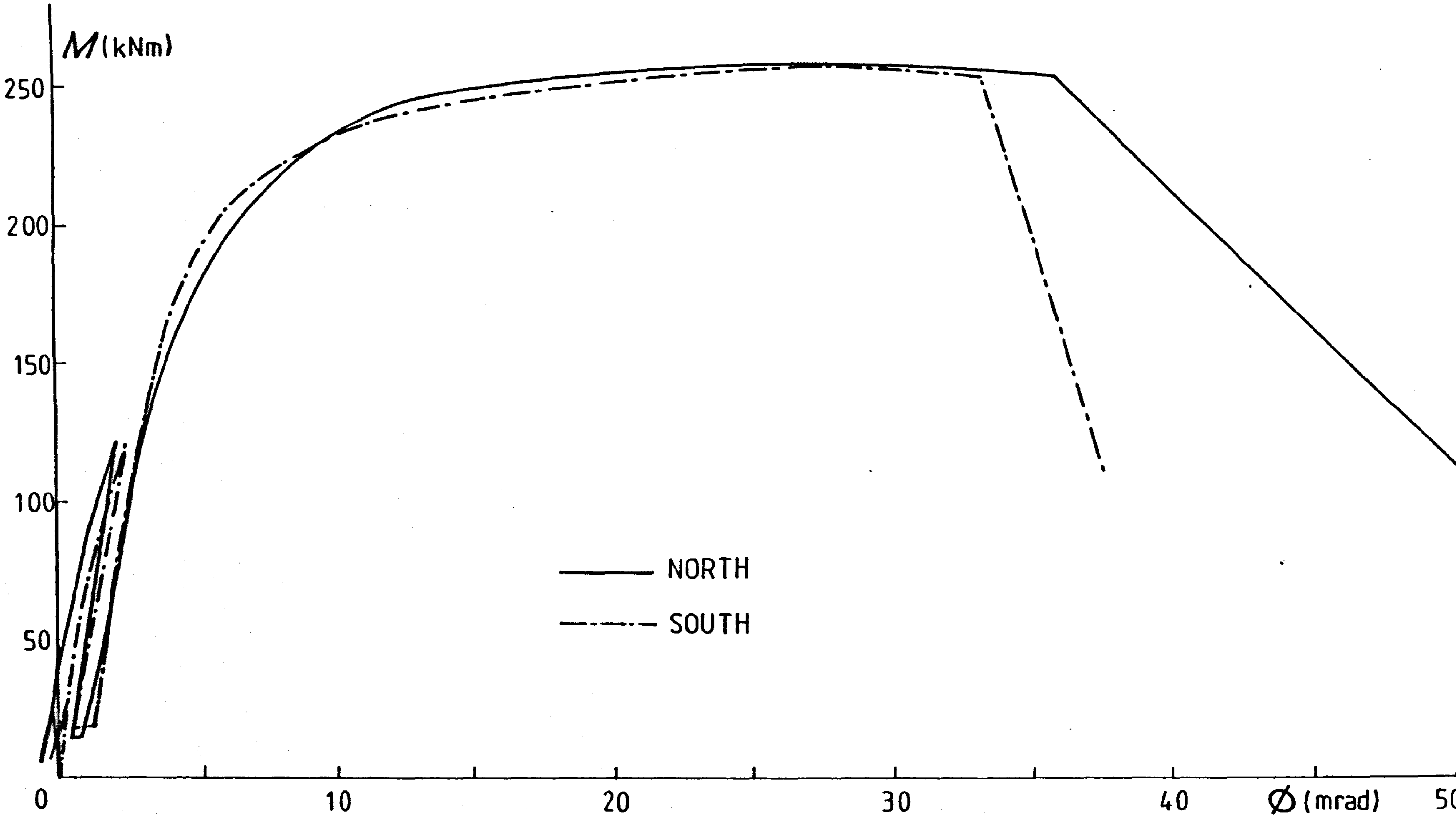


Fig. 5-12, Moment-rotation curve of Test 1 measured by transducers

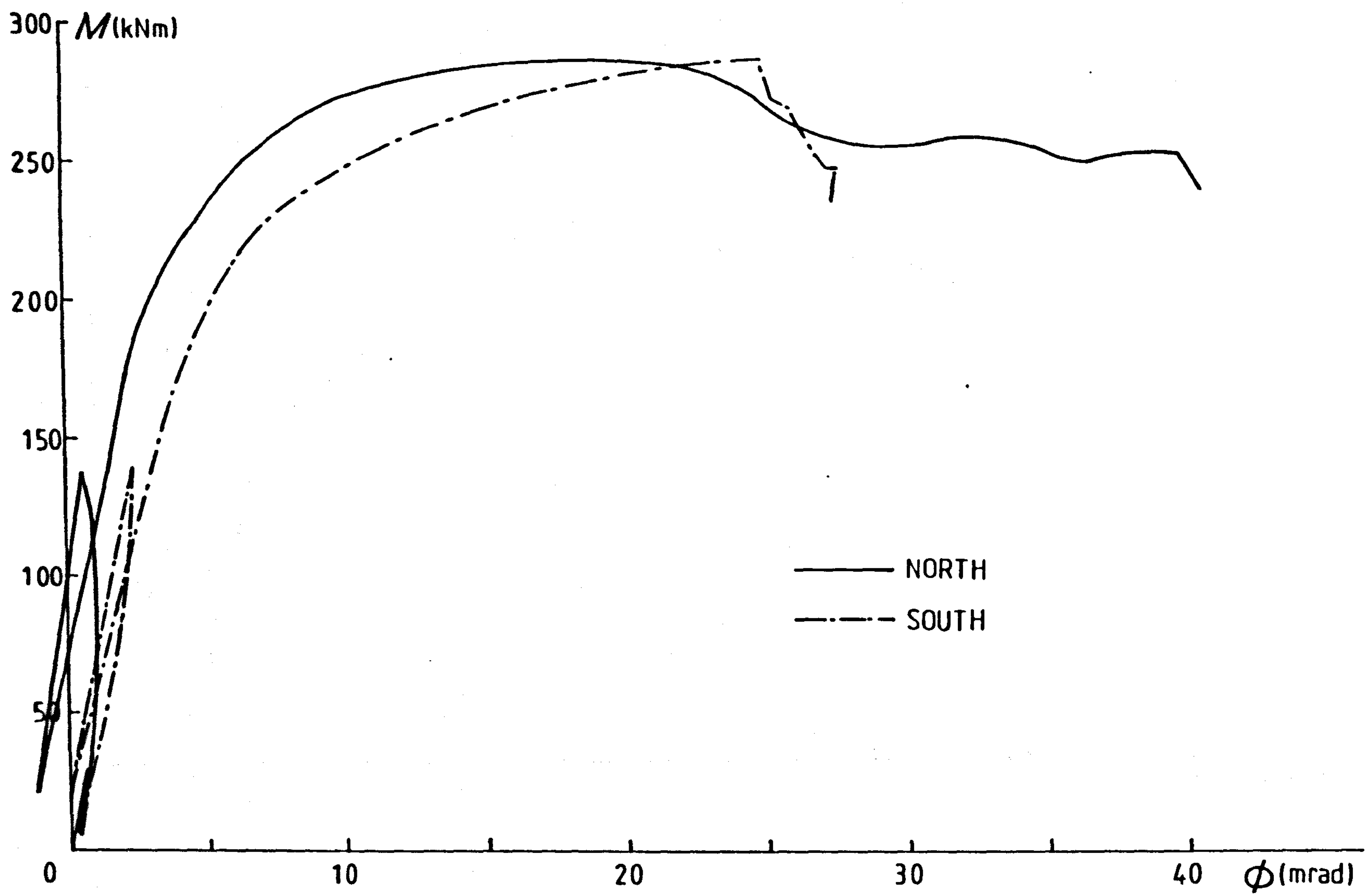


Fig. 5-13, Moment-rotation curve of Test 2 measured by transducers

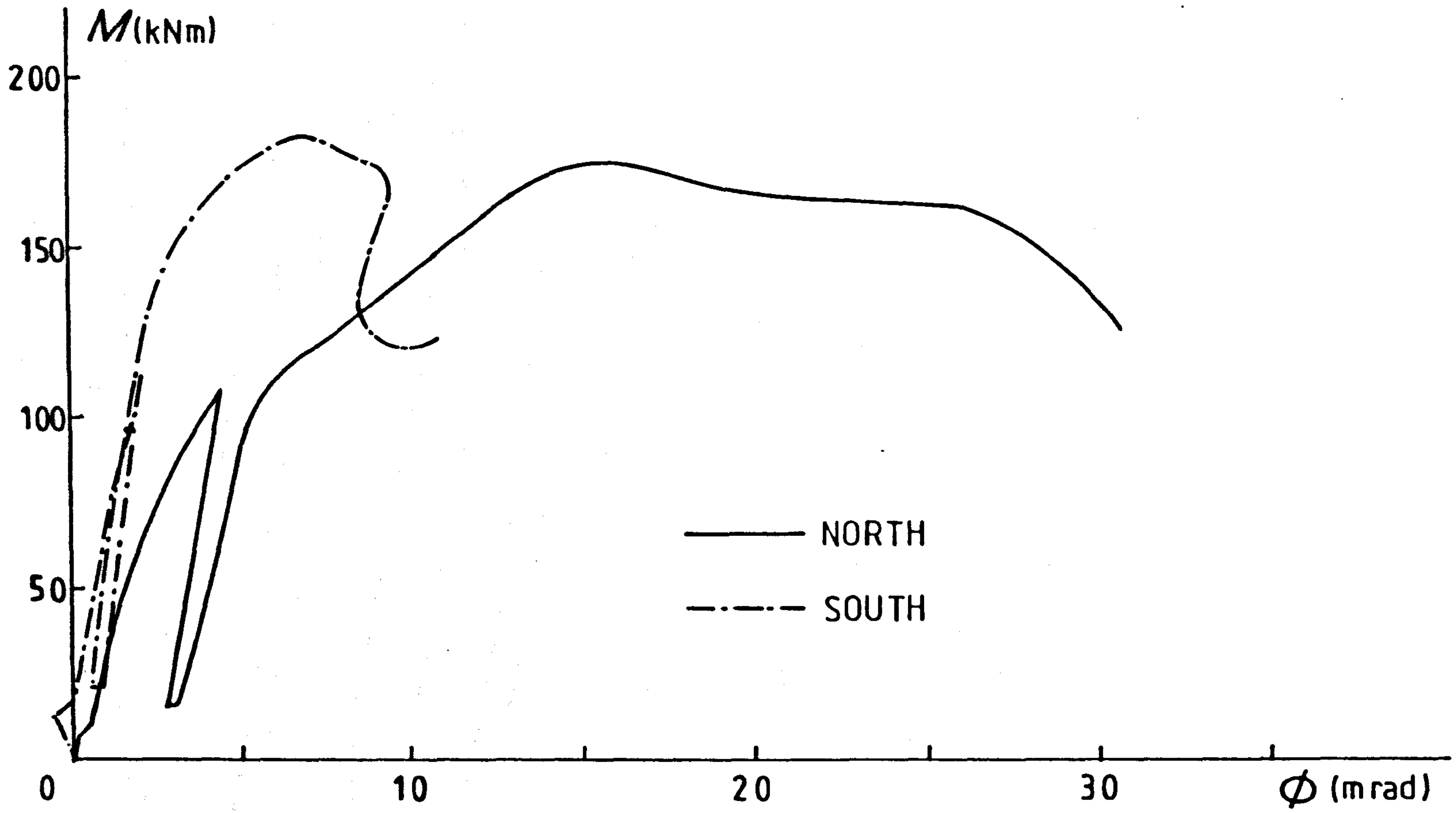


Fig. 5-14, Moment-rotation curve of Test 3 measured by transducers

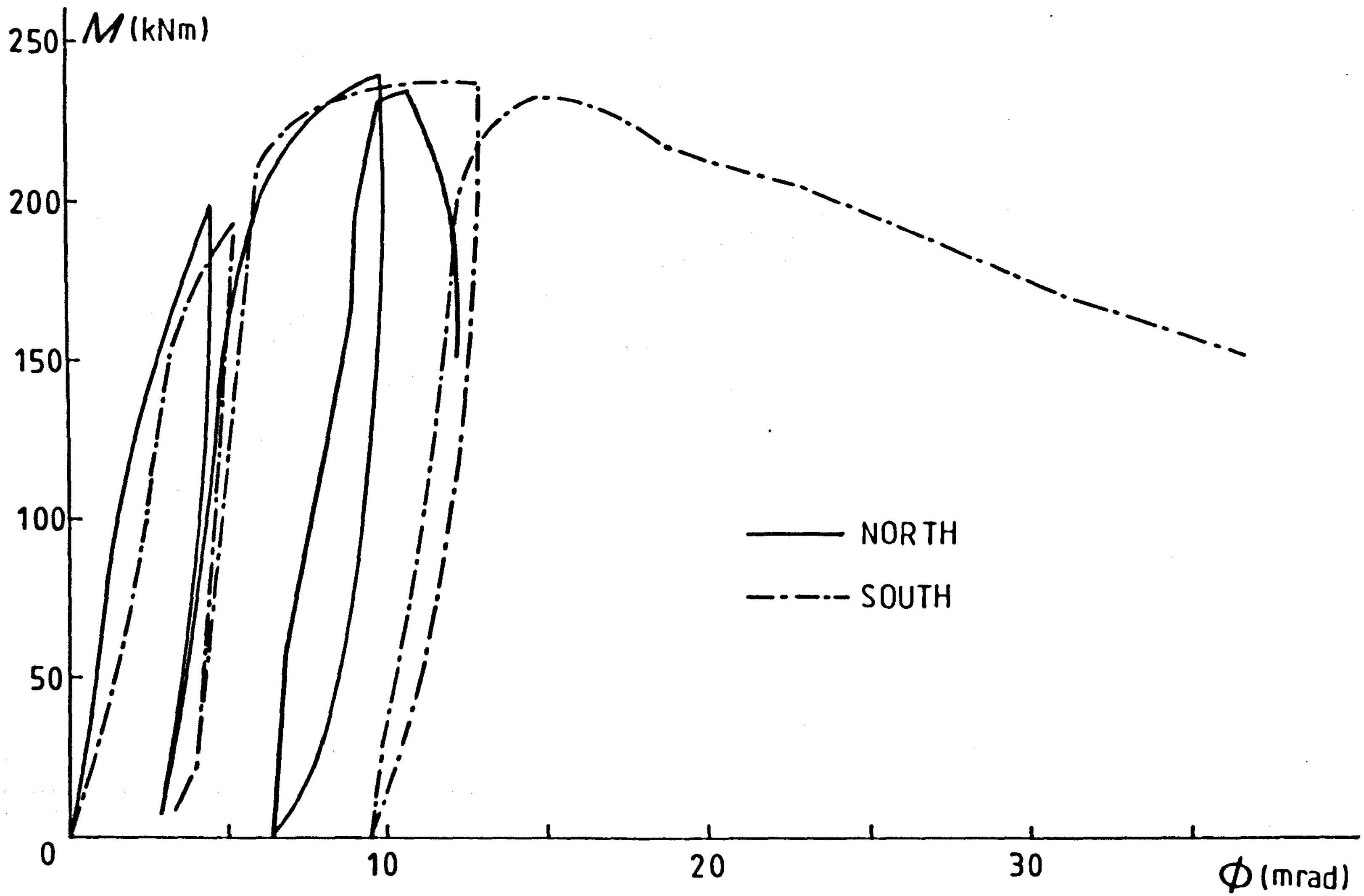


Fig. 5-15, Moment-rotation curve of Test 4 measured by transducers

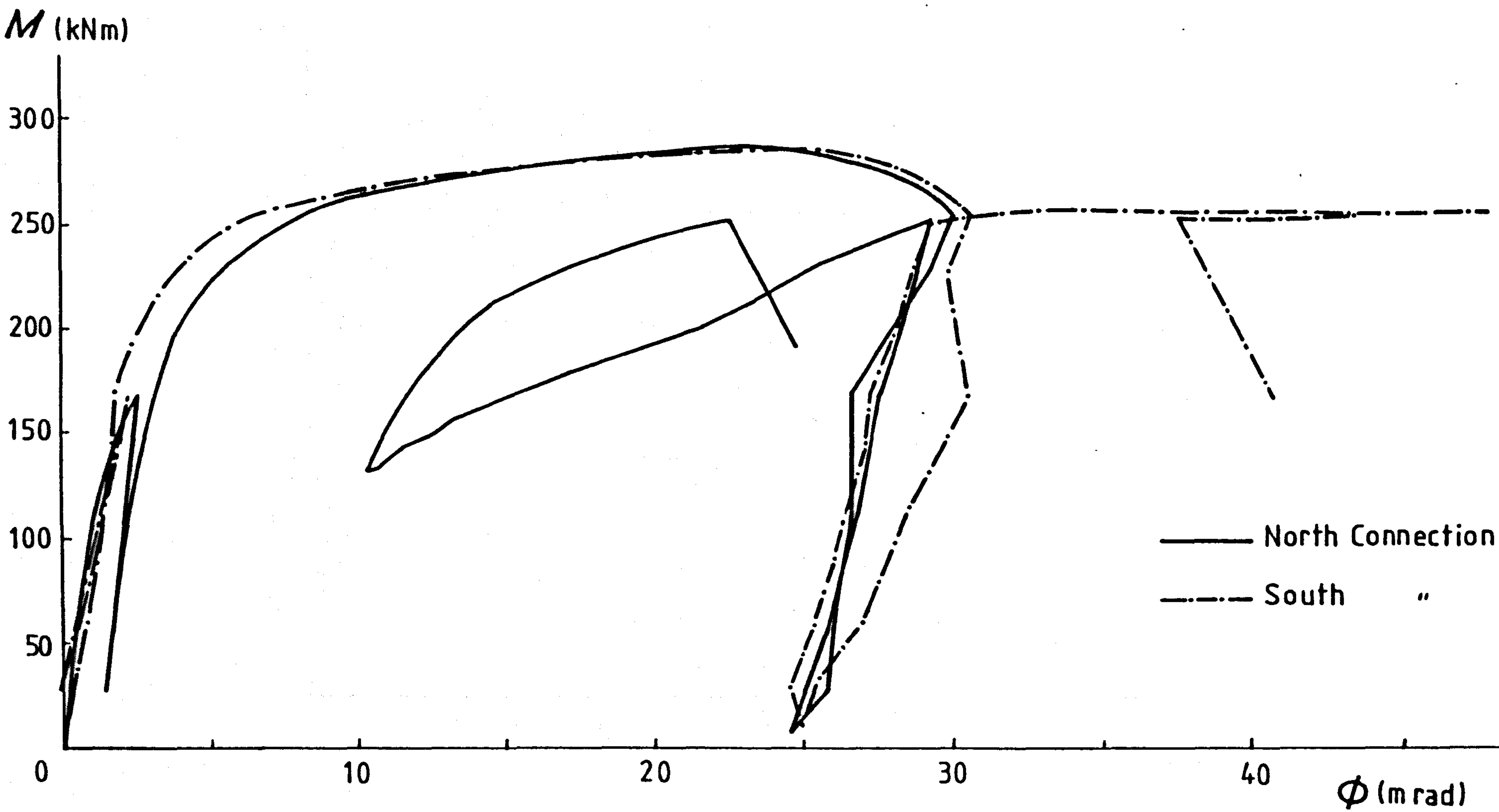


Fig. 5-16,Moment-rotation curve of Test 5 measured by electronic inclinometers

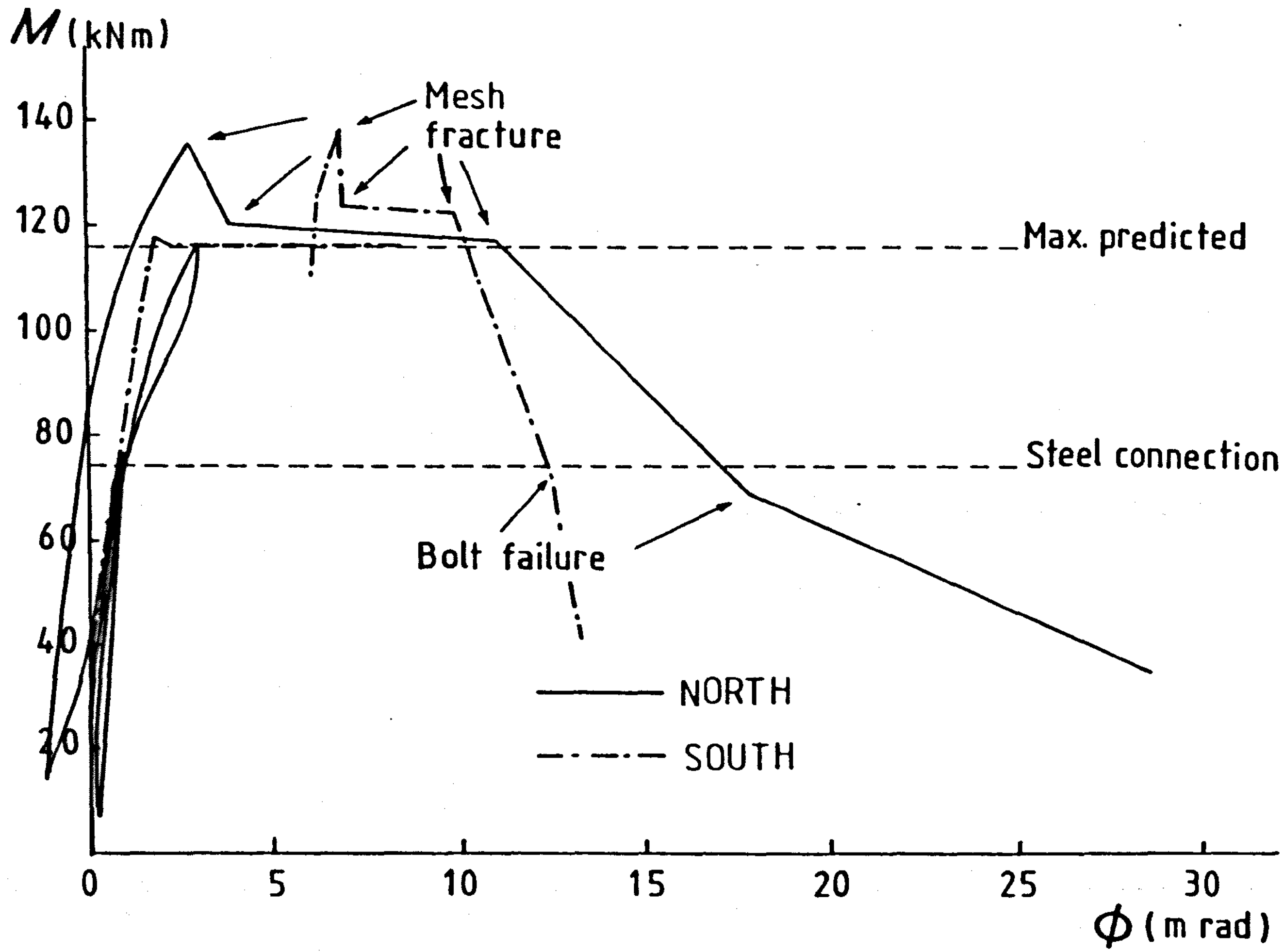


Fig. 5-17,Moment-rotation curve of Test 6 measured by electronic inclinometers

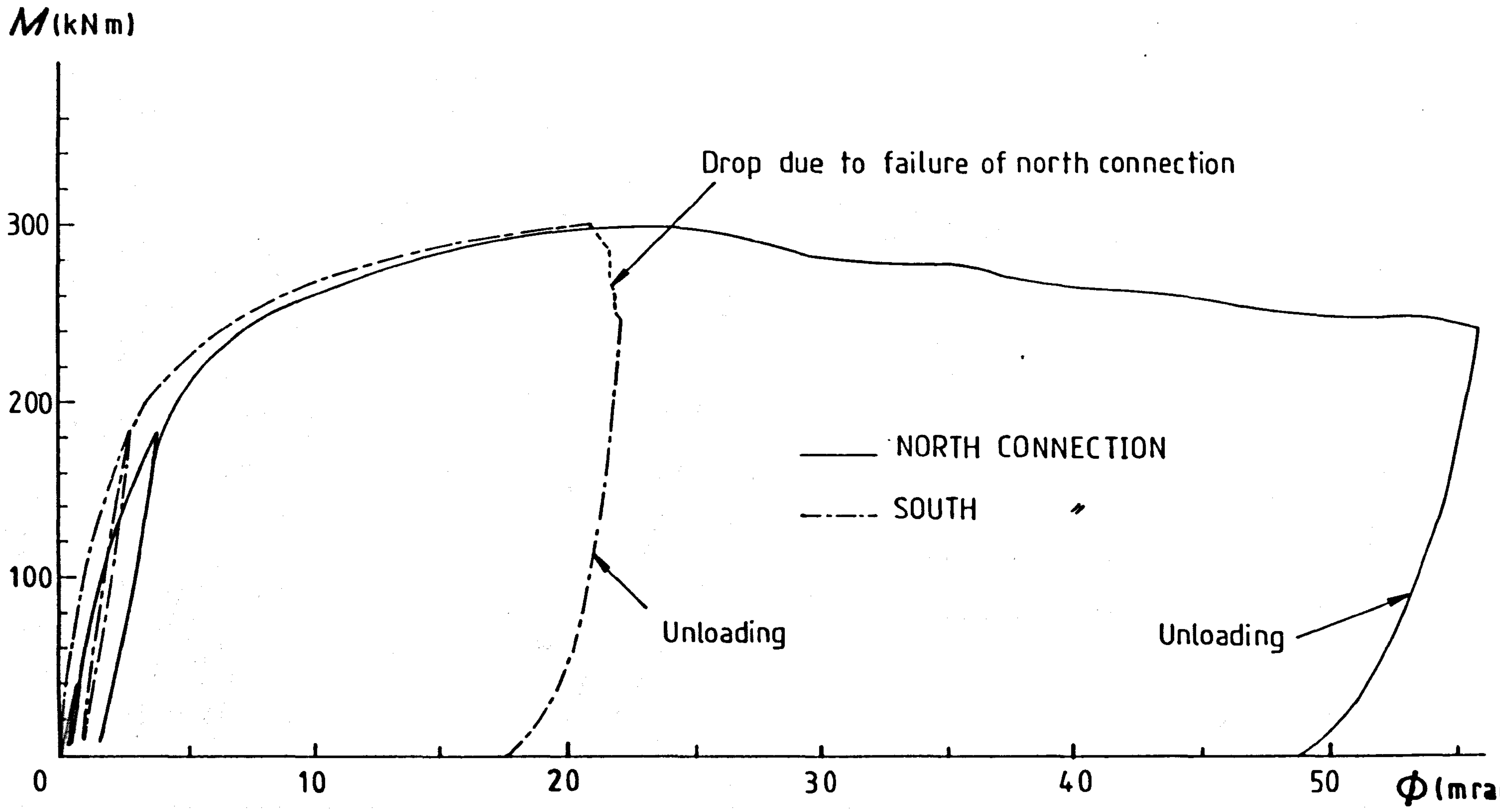


Fig. 5-18, Moment-rotation curve of Test 7 measured by electronic inclinometers

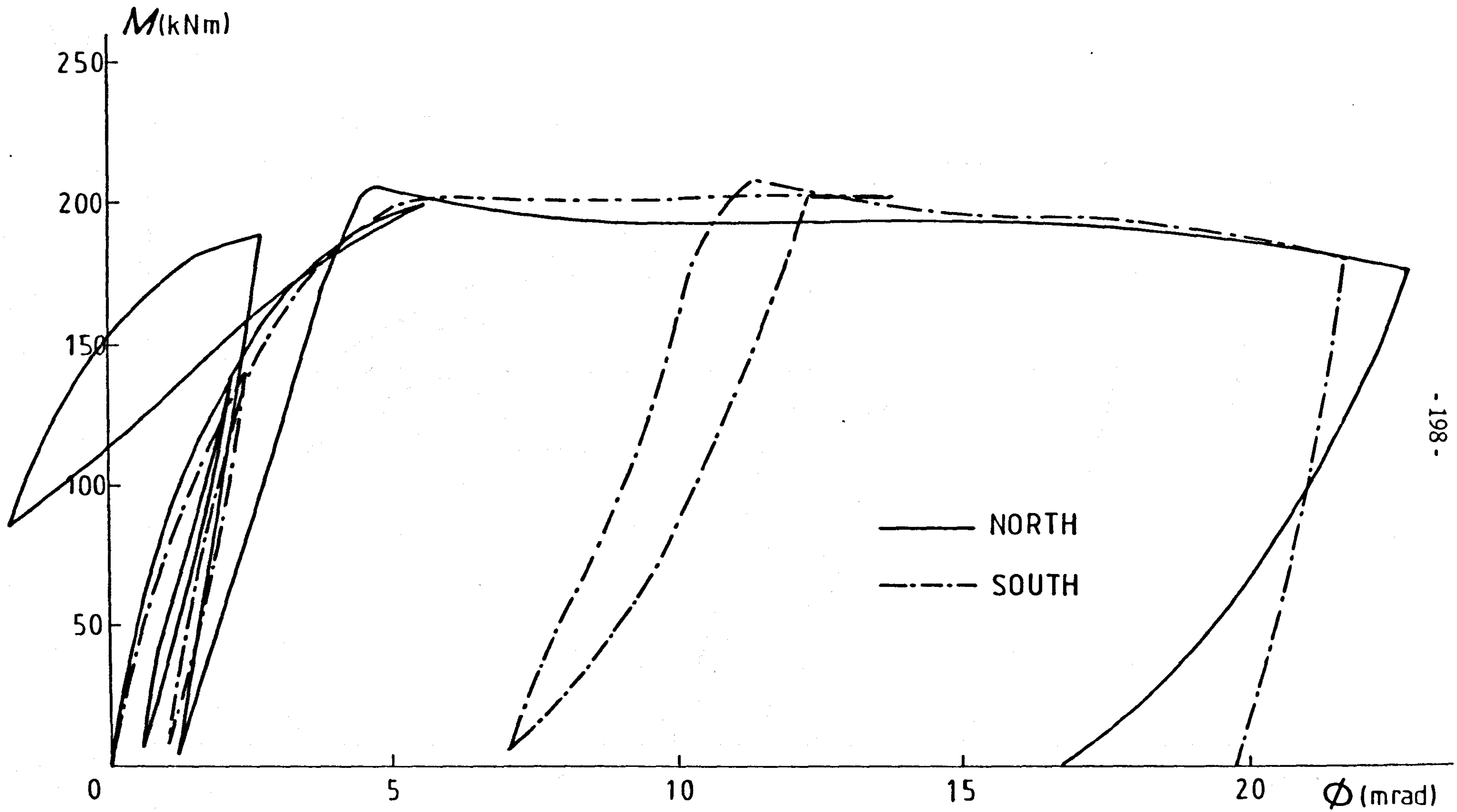


Fig. 5-19,Moment-rotation curve of Test 8 measured by electronic inclinometers

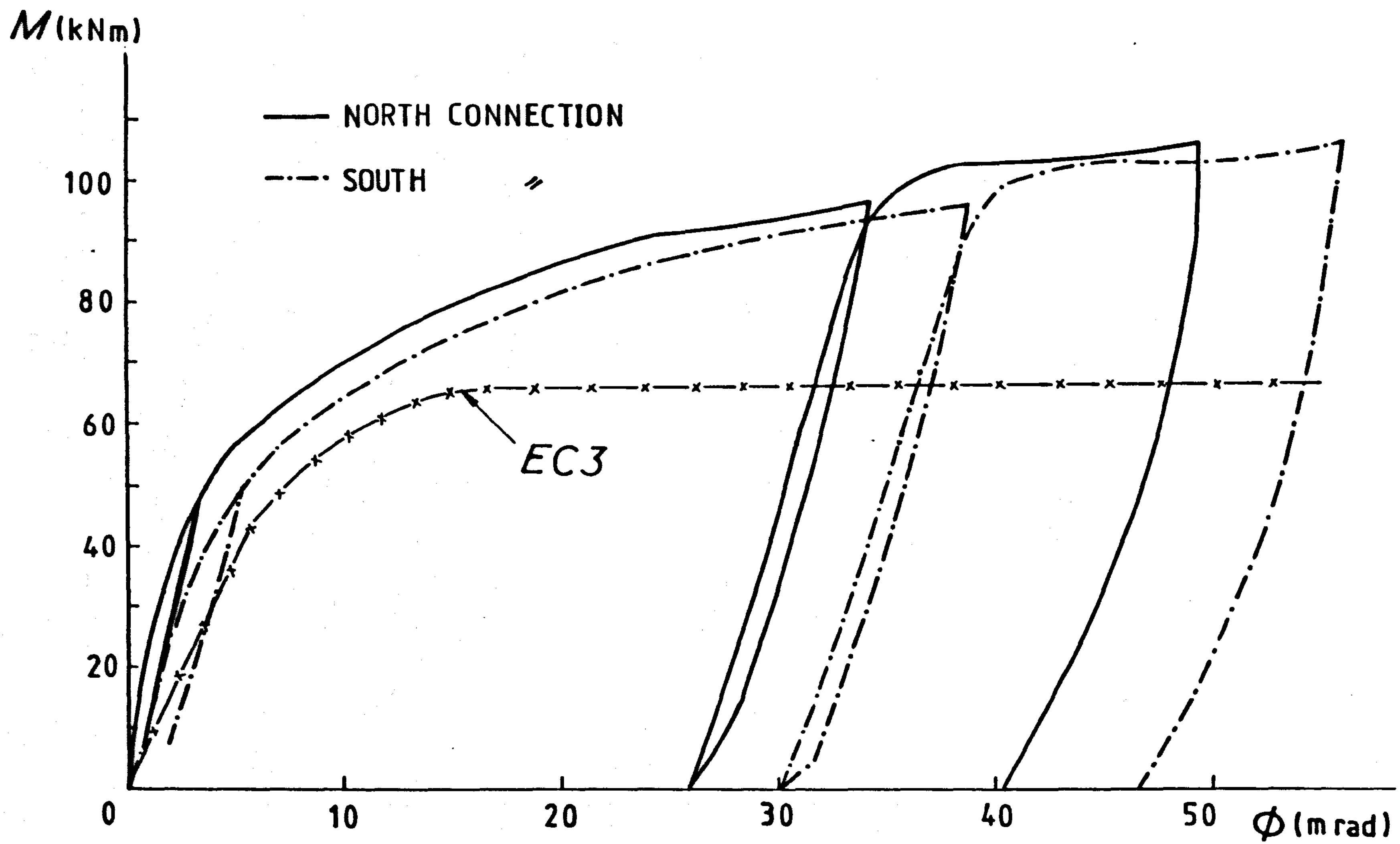


Fig. 5-20,Moment-rotation curve of Test 9 measured by electronic inclinometers

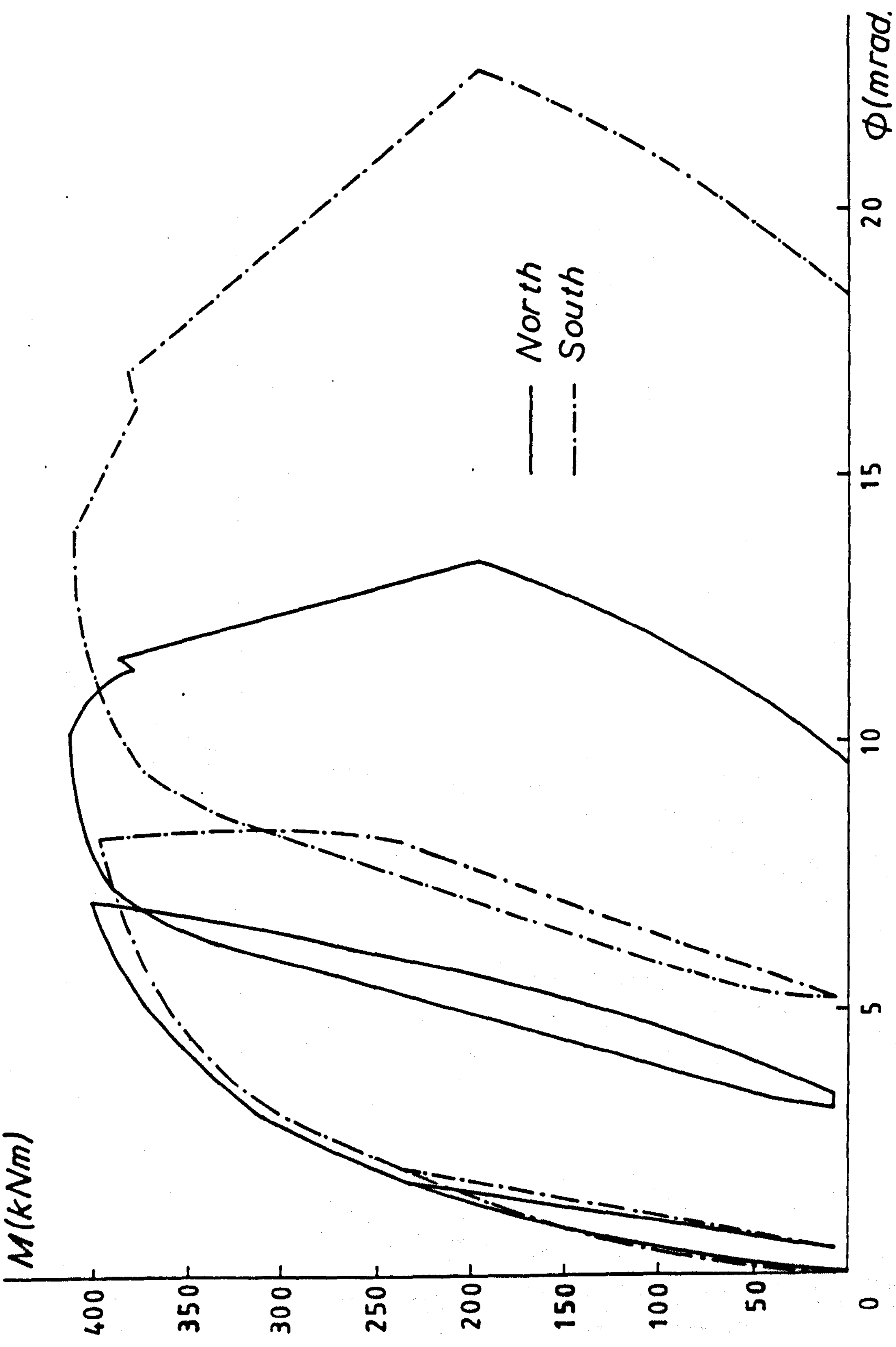


Fig.5-21 Moment-rotation curve of Test 10 measured by electronic inclinometers

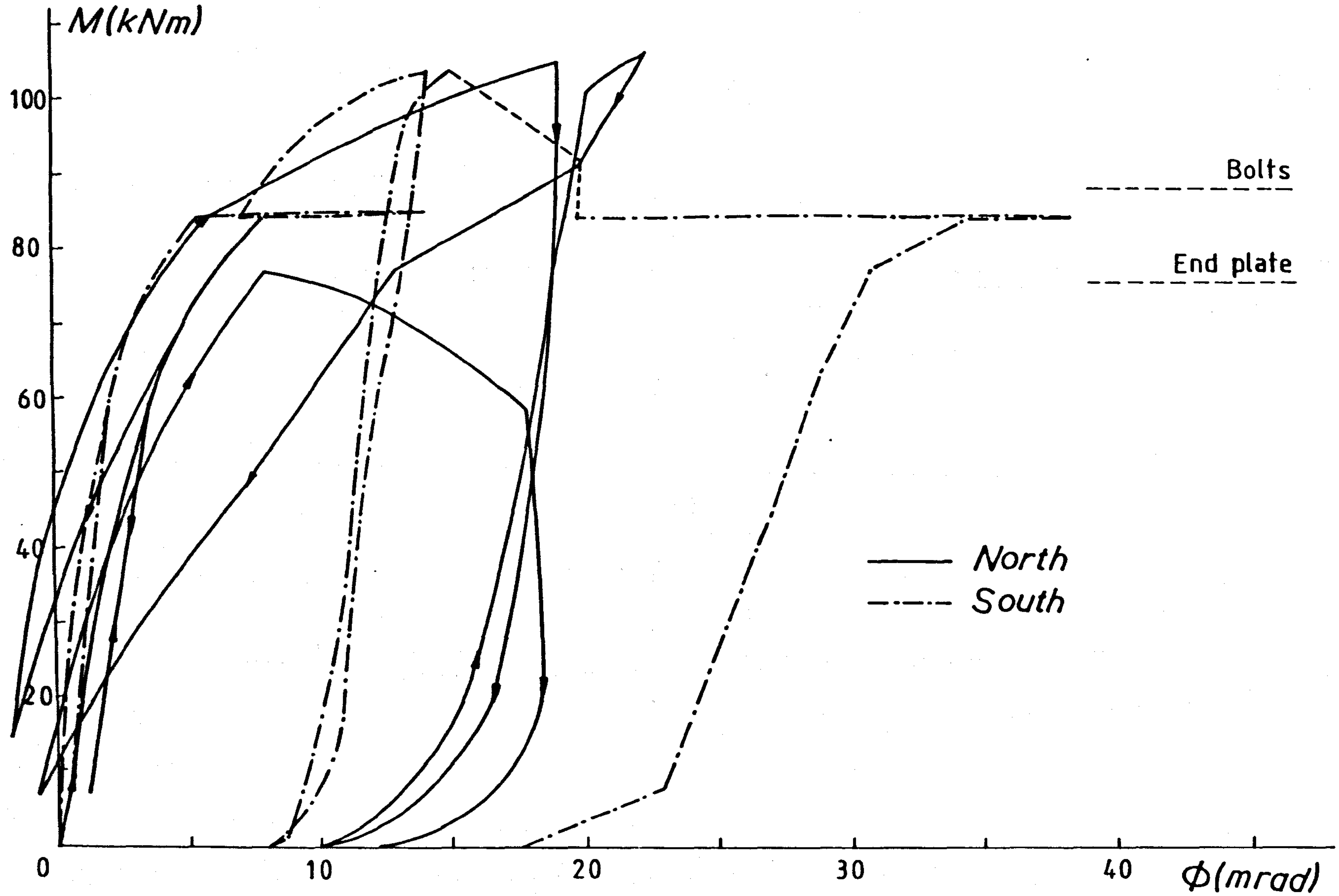


Fig.5-22,Moment-rotation curve of Test 11 measured by electronic inclinometers

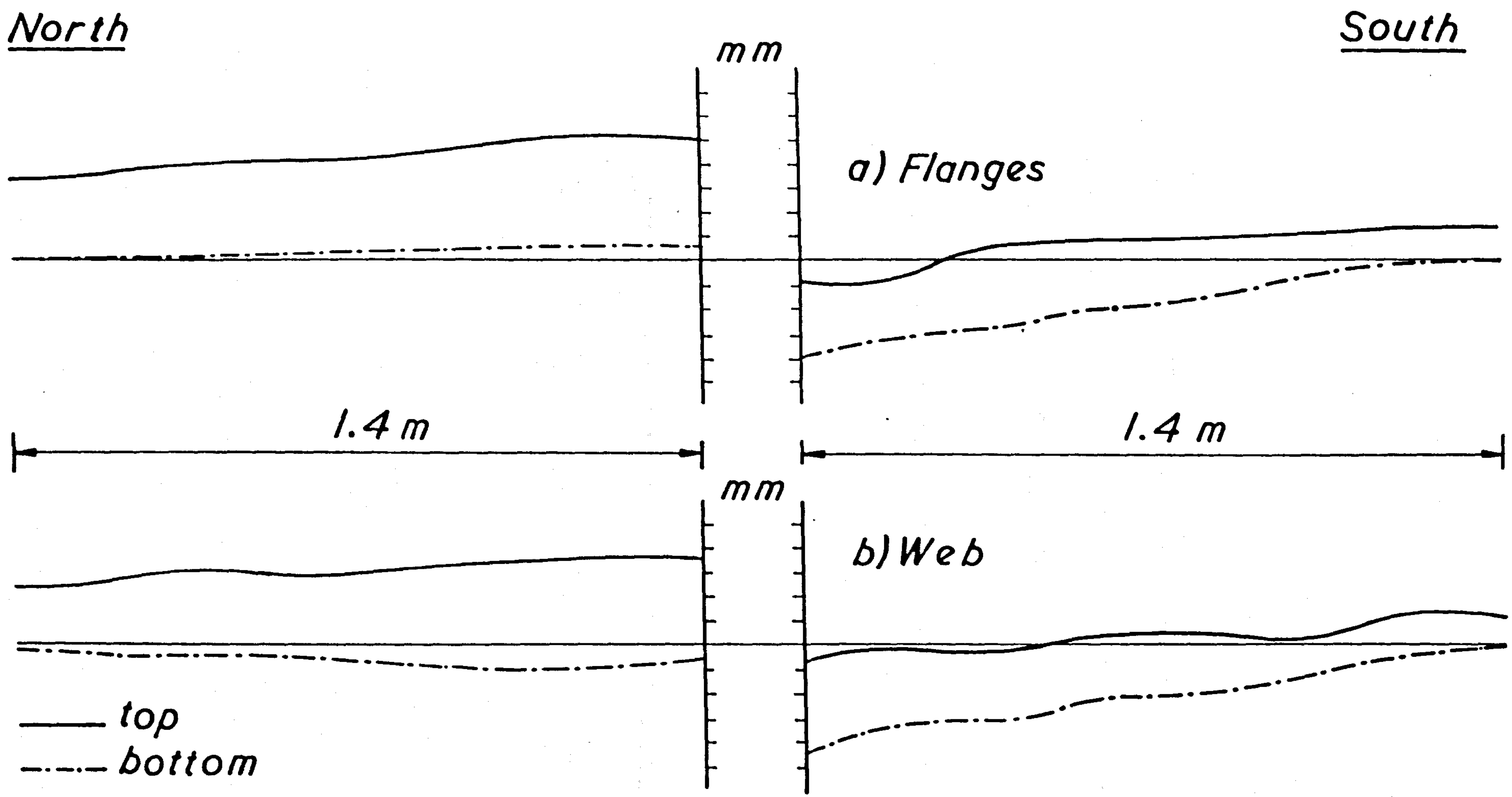


Fig. 5-23, Initial imperfection of steel beam in Test 2 (measured in respect to a line connecting two end corners of bottom flanges of cantilevers)

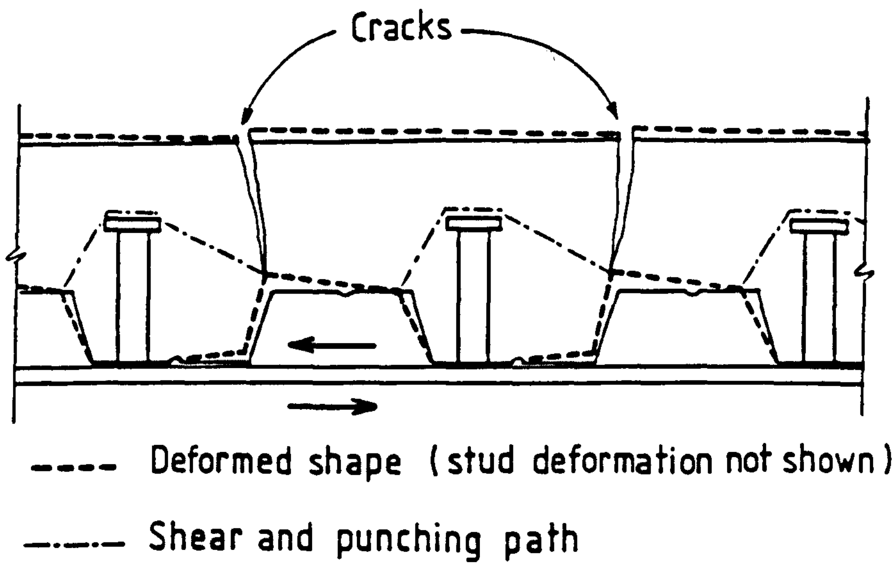


Fig. 5-24, Deformed shape of profiled steel sheeting

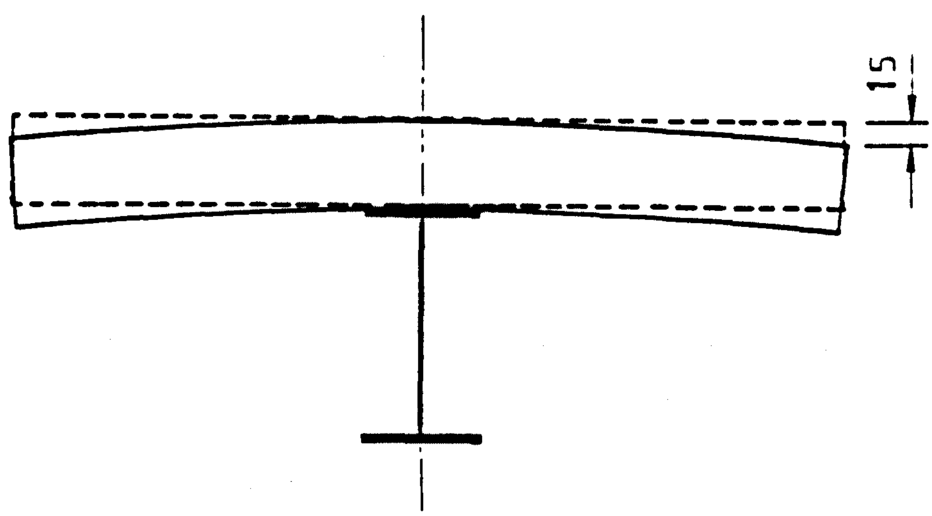


Fig. 5-25, Transverse bending of concrete slab in Test 7

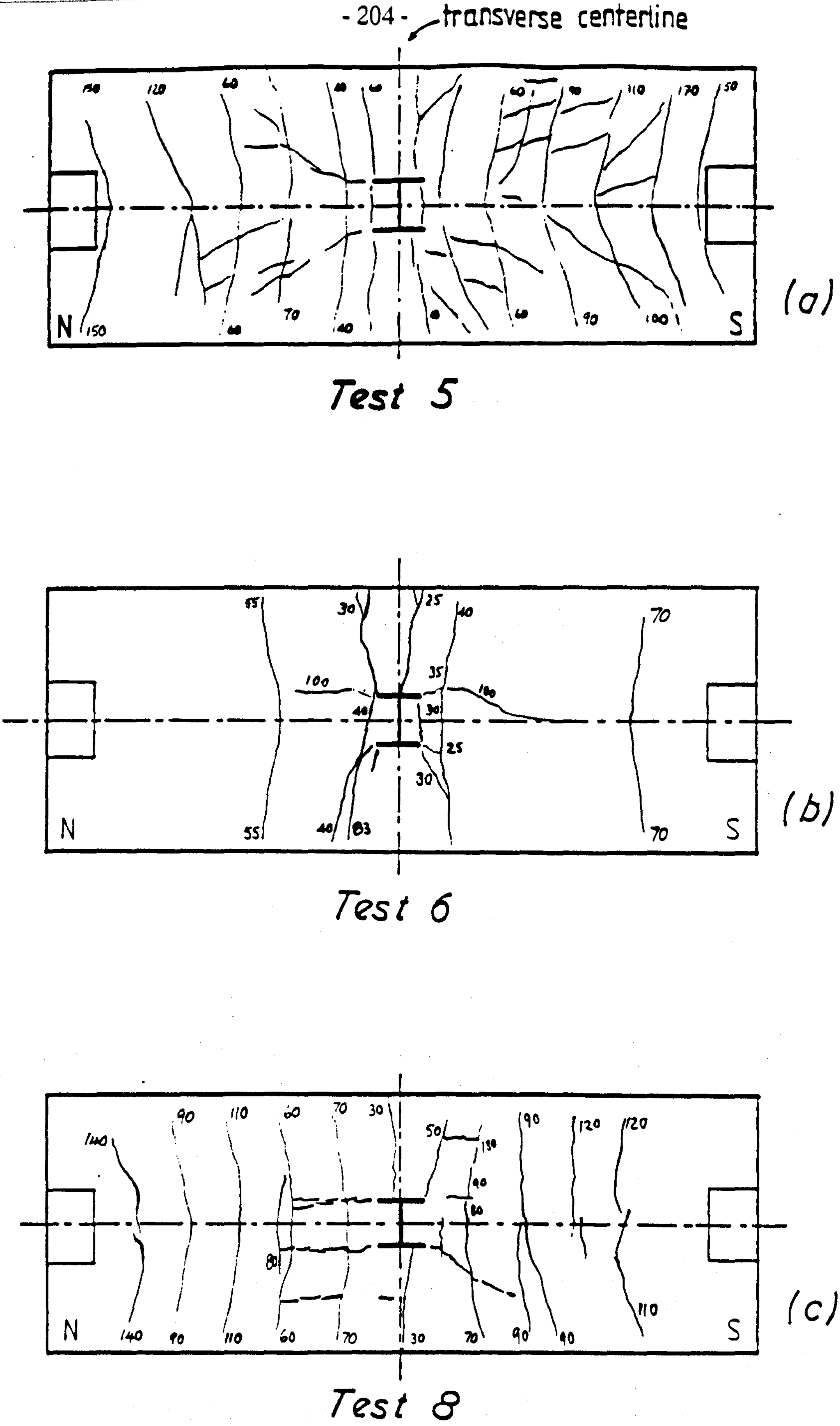
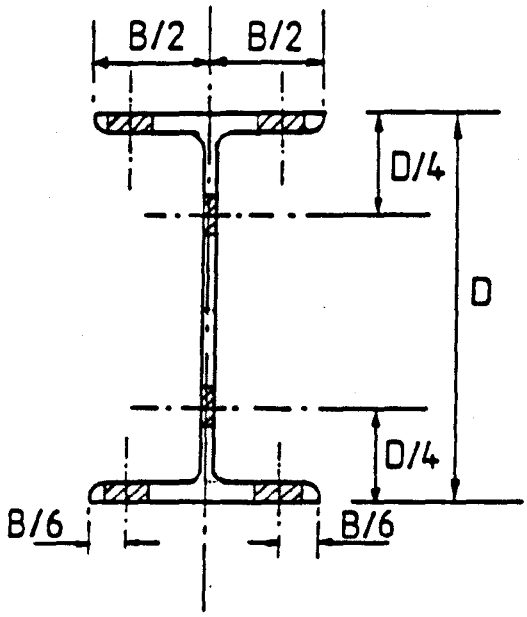
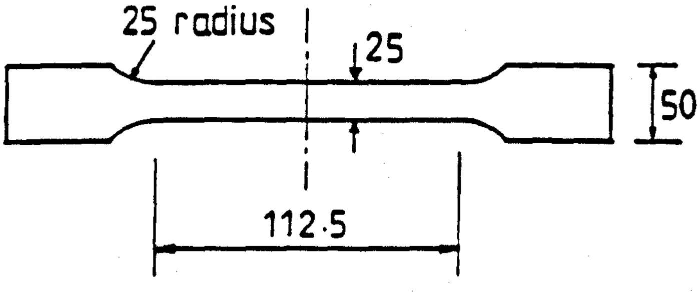


Fig. 5-26, Formation of cracks in minor axis tests



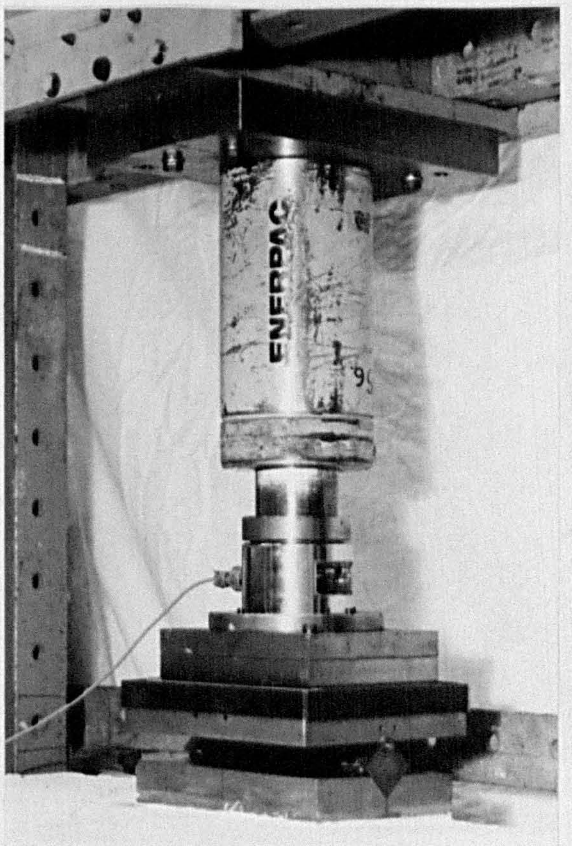
Coupon positions



Test specimen

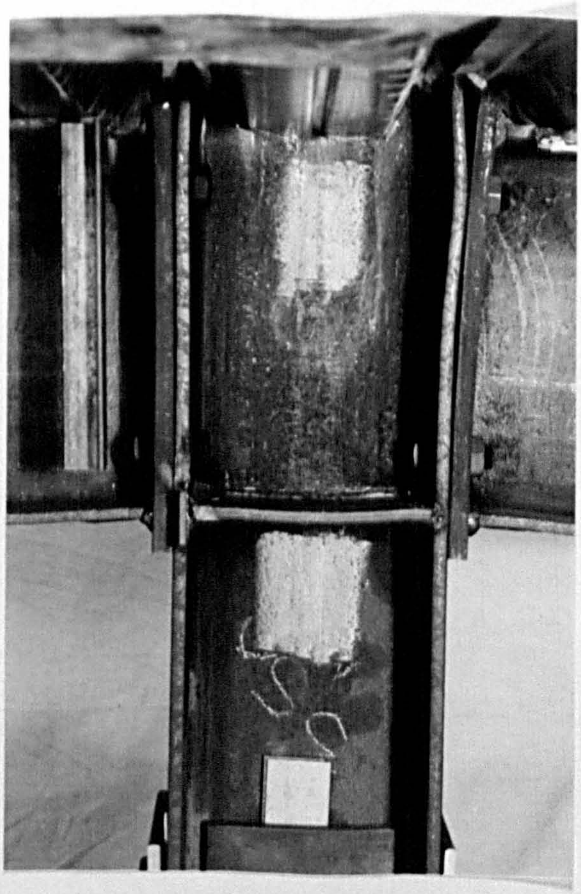
FIG. 5-27, Positions and dimentions of coupon samples

(a) Instrumentation and data aquisition system

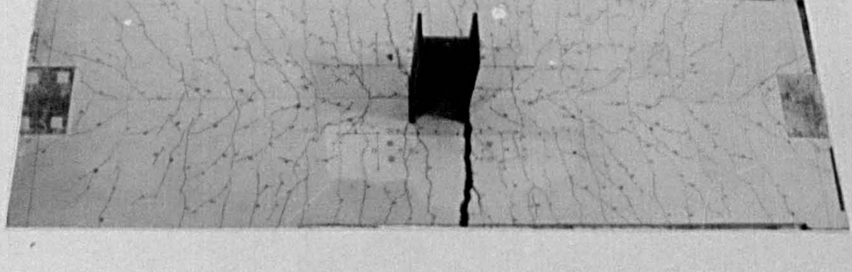


(b) Arrangement of jack, load cell, rollers and knife edge

Plate 1, Data aquisition system and loading arrangement

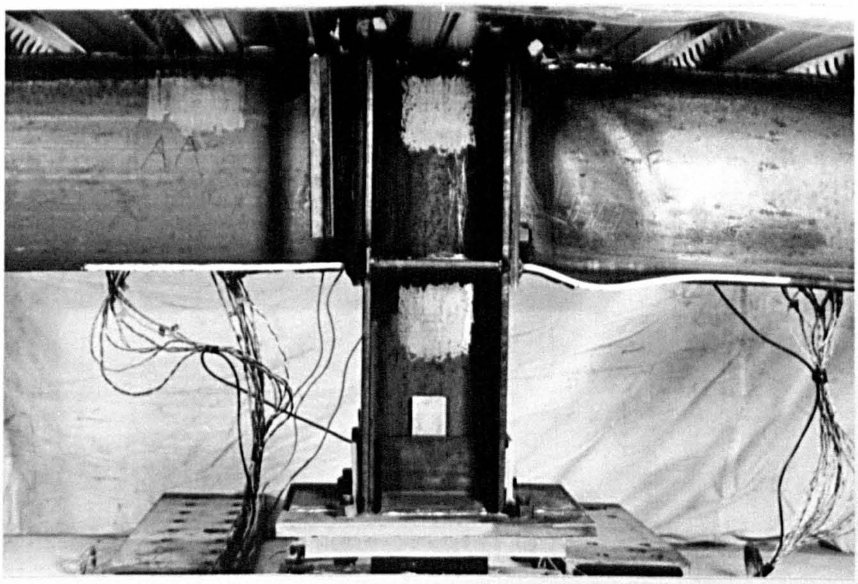


(a) Deformation of Connection (Test 1)

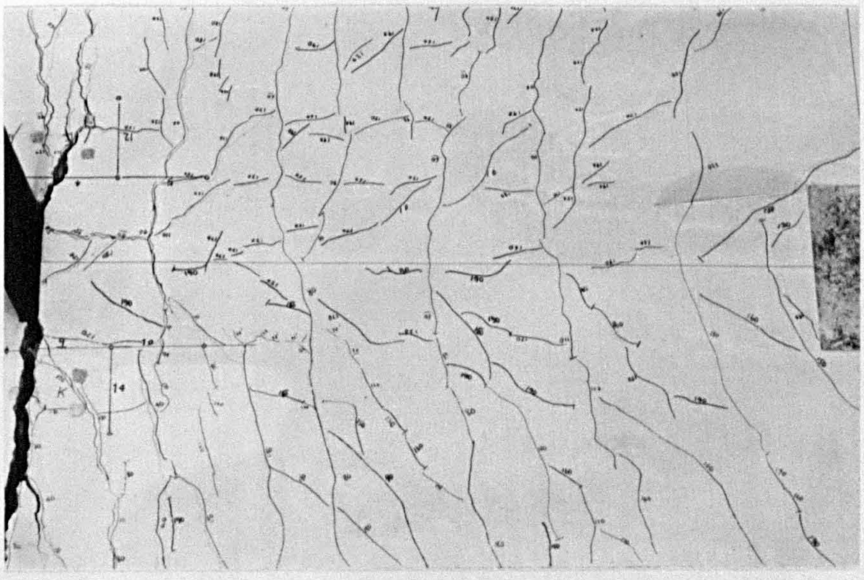


(b) Crack pattern (Test 1)

Plate 2, Deformation of connection and crack pattern in concrete slab (Test 1)

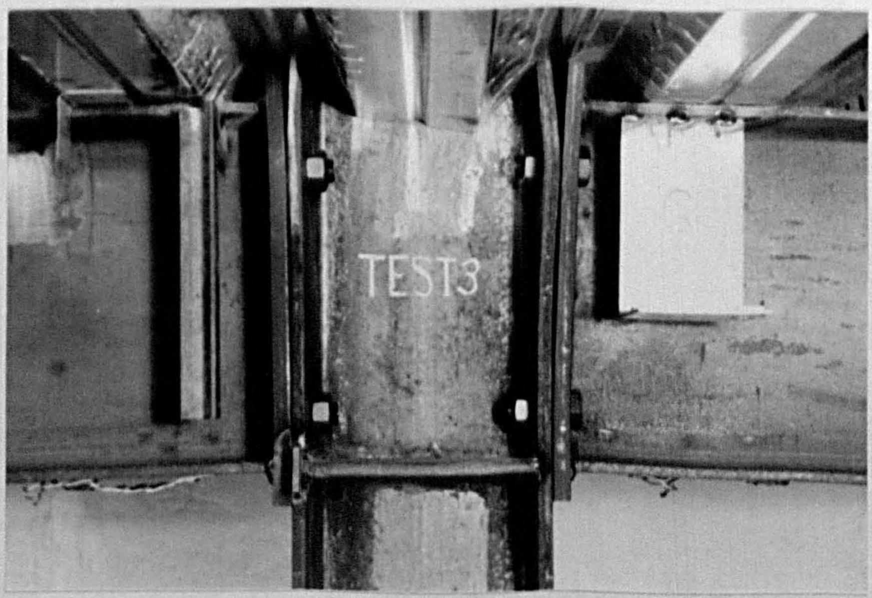


(a) Deformation of connection (Test 2)



(b) Crack pattern (Test 2, North side)

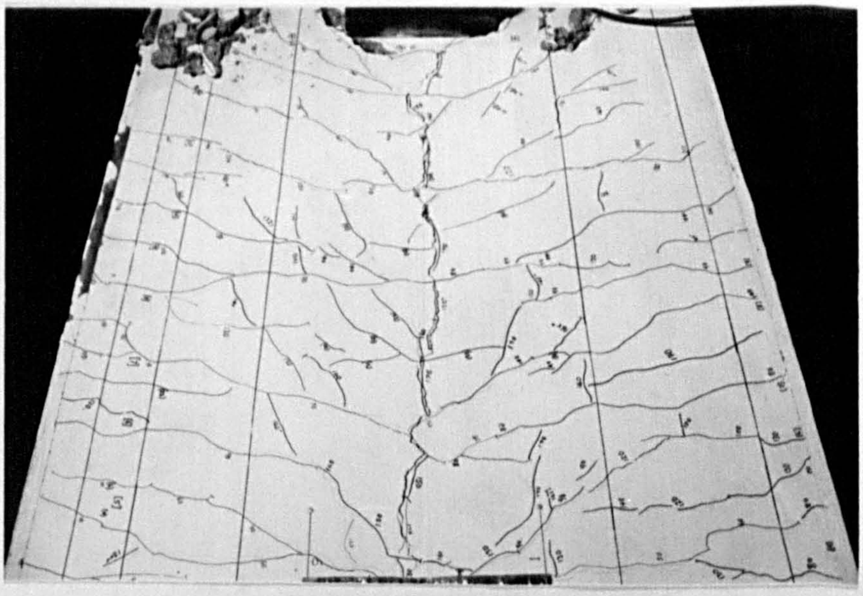
Plate 3, Deformation of connection and crack pattern in concrete slab (Test 2)



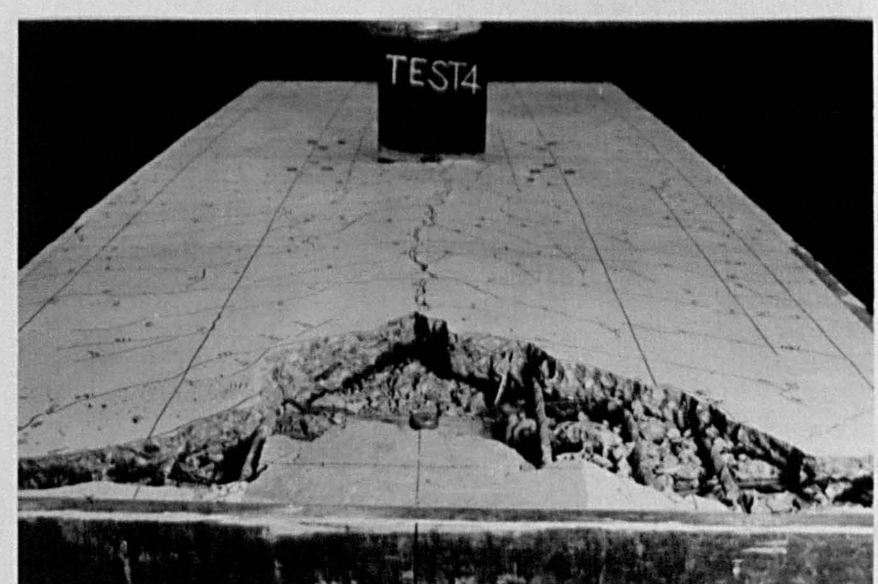
(a) Deformation of connection (Test 3)

(b) Crack pattern (Test 3, North side)

Plate 4, Deformation of connection and crack pattern in concrete slab (Test 3)



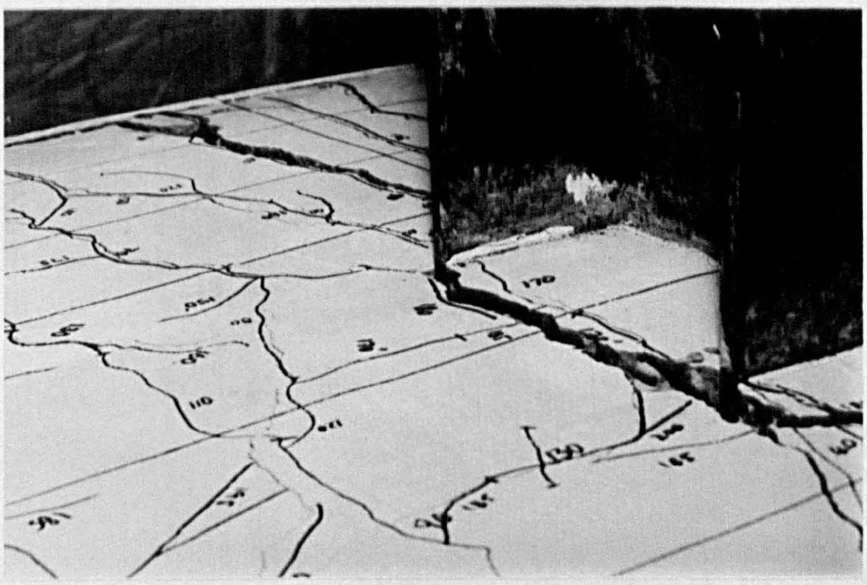
(a) Crack pattern (Test 4, South side)



(b) Uplift of concrete slab (Test 4, South side)

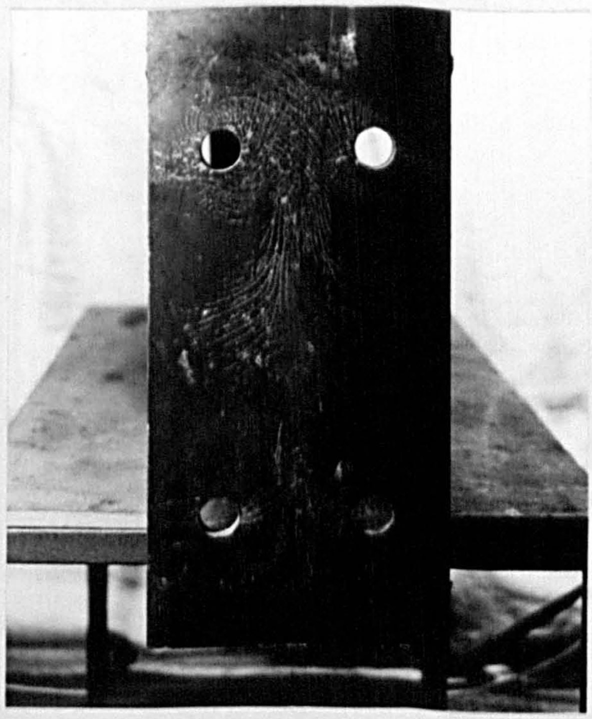
Plate 5, Crack pattern and uplift of concrete slab (Test 4)

(a) Local buckling of steel beam (Test 5)

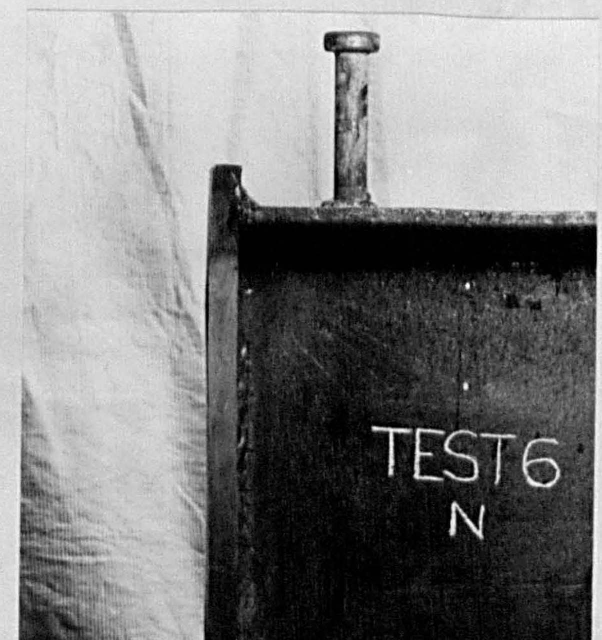


(b) Cracks at failure (Test 5)

Plate 6, Local buckling of steel beam and concrete cracks at failure (Test 5)

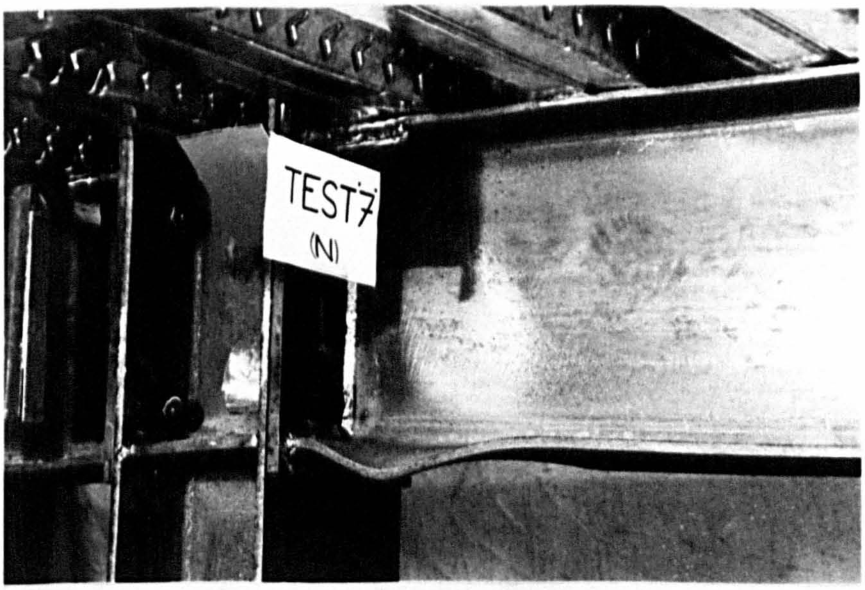


(a) Signs of strain in end plate (Test 6)

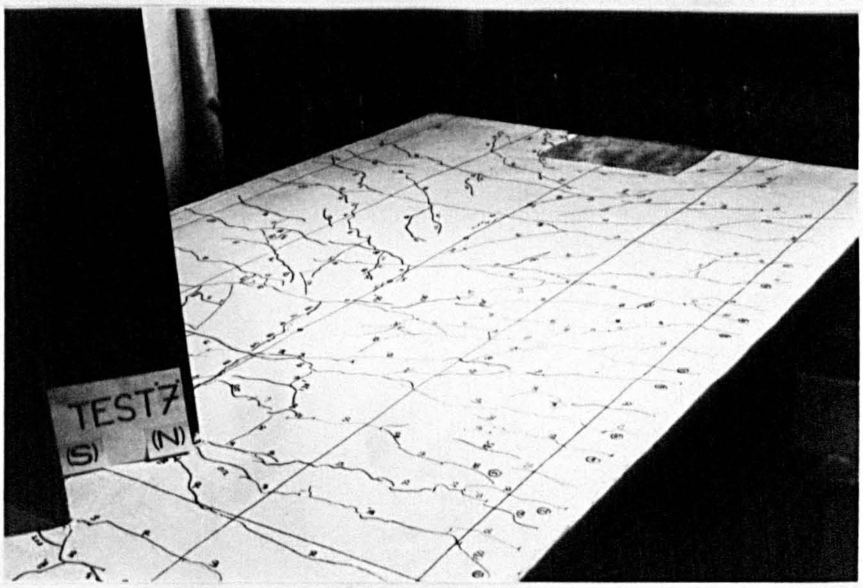


(b) Deformation of end plate (Test 6)

Plate 7, Signs of strain and deformation of end plate (Test 6)

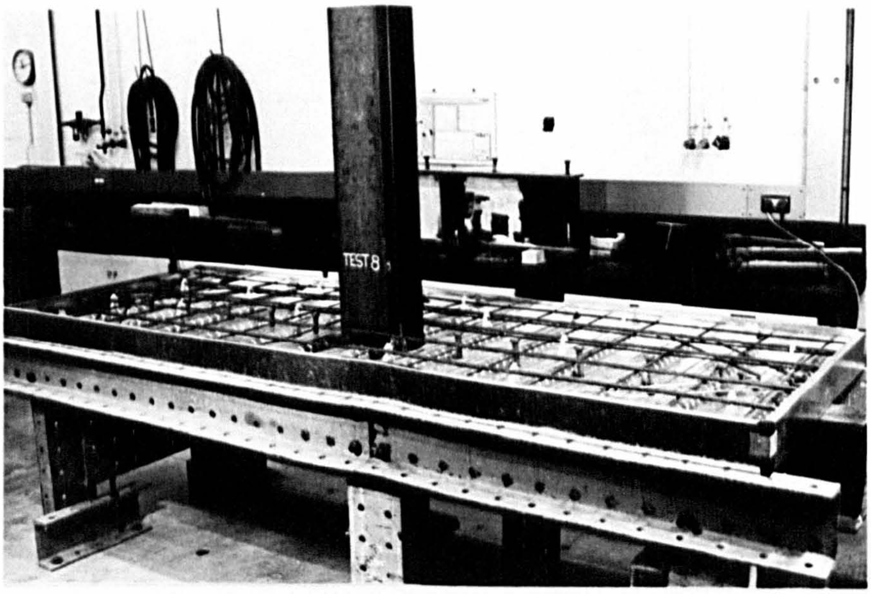


(a) Local buckling of beam flange and web (Test7, North side)

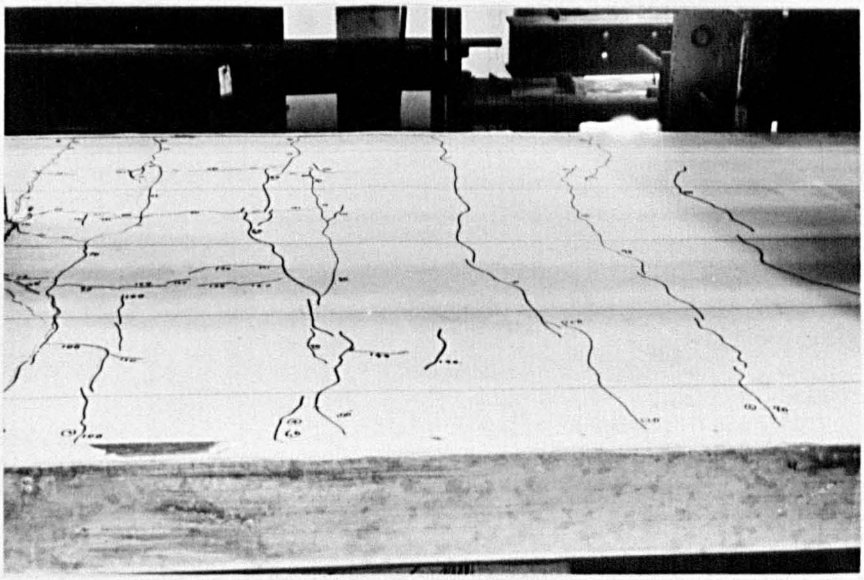


(b) Crack pattern (Test 7, North side)

Plate 8, Buckling of steel beam and crack pattern in concrete slab (Test 7)

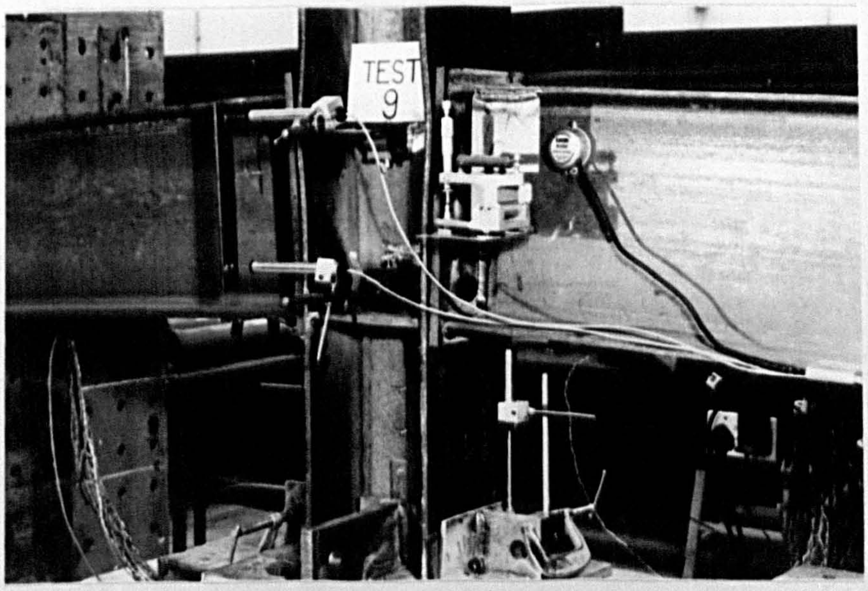


(a) Specimen before casting (Test 8)



(b) Crack pattern (Test 8)

Plate 9, Specimen before casting and crack pattern in concrete slab (Test 8)

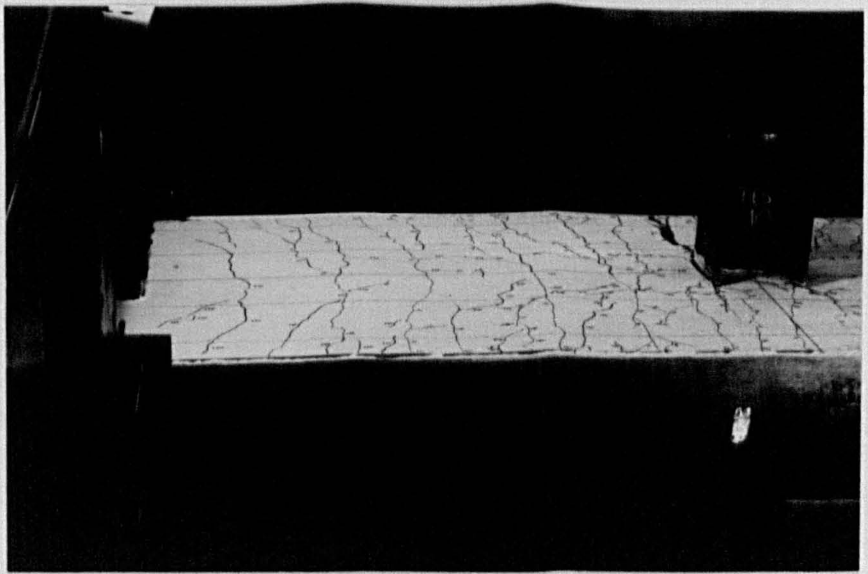


(a) Rotation measurement (Test 9)

(b) Deformation of connection (Test 9)

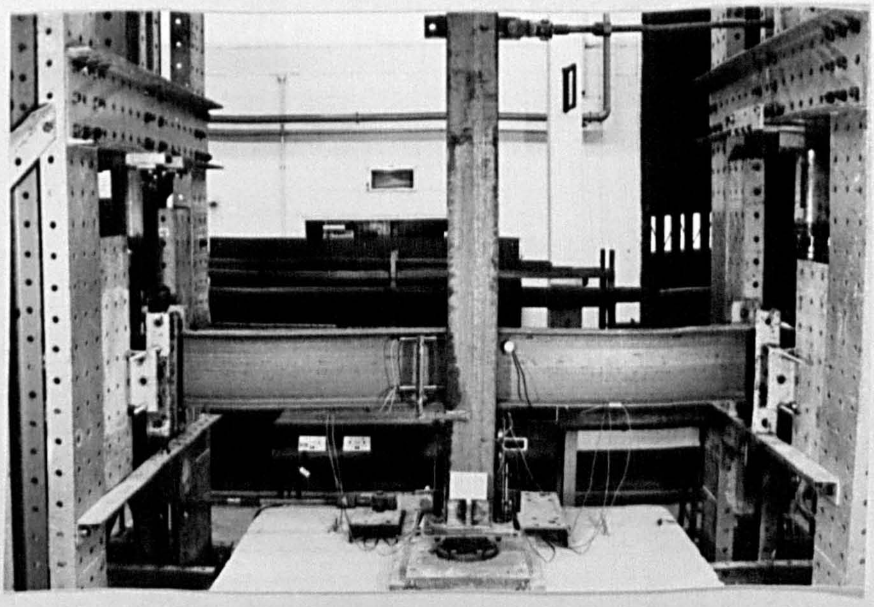
Plate 10, Rotation measurement and deformation of connection (Test 9)

(a) Deformation of connection (Test 10)



(b) Crack pattern (Test 10)

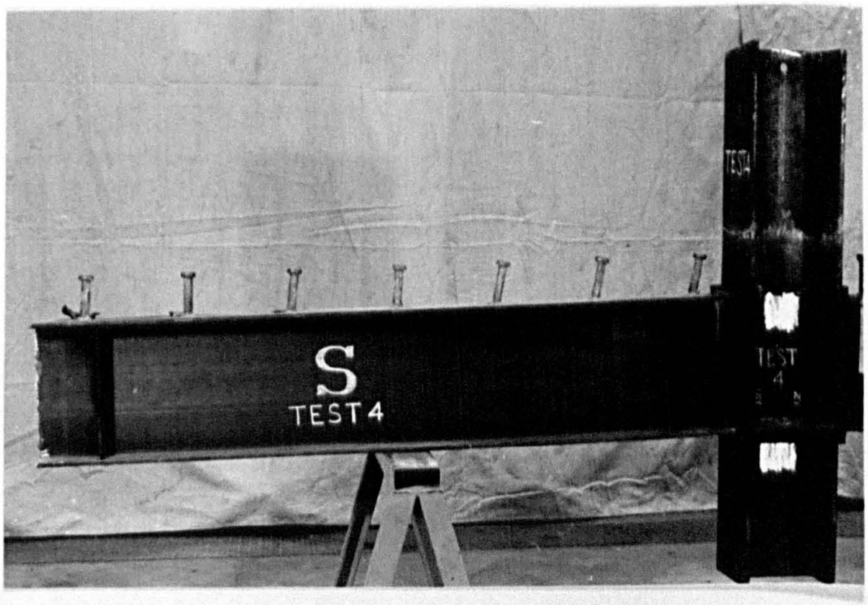
Plate 11,Deformation of connection and crack pattern in concrete slab(Test 10)



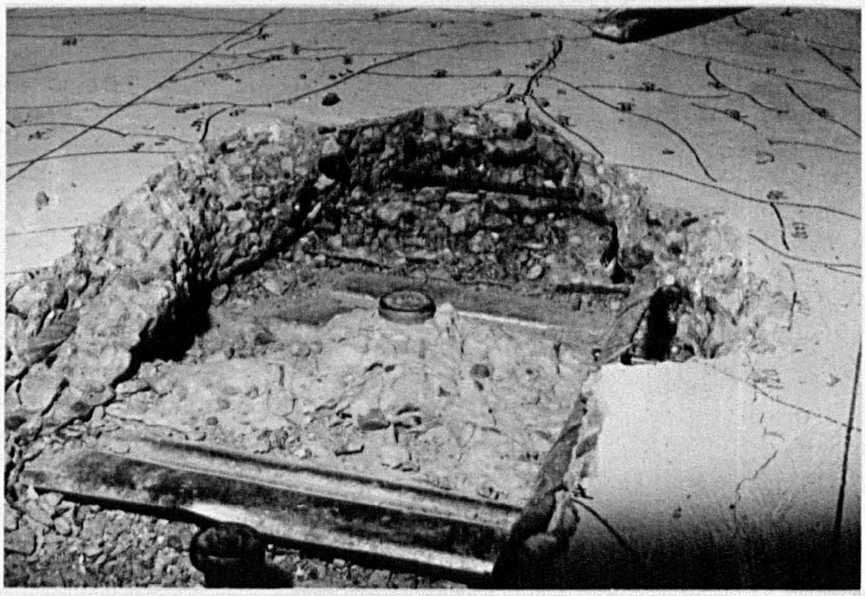
(a) Minor axis bare steel specimen during test (Test 11)

(b) Signs of strain in end plate (Test 11)
(Note deformation around holes of top row of bolts)

Plate 12, Minor axis bare steel specimen and end plate after test (Test 11)



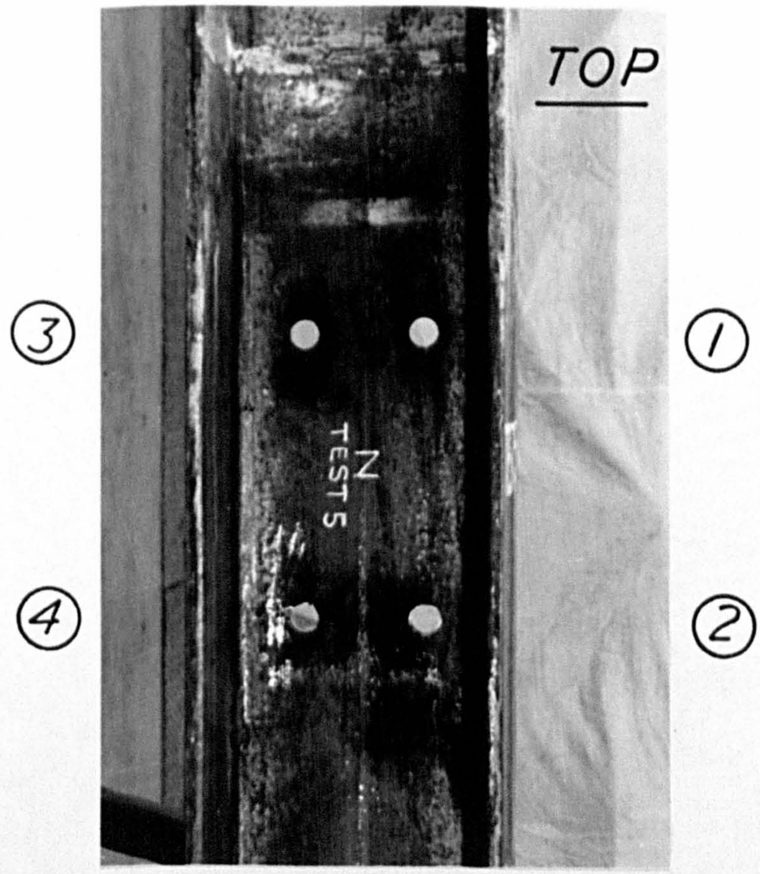
(a) Specimen after removal of concrete slab (Test 4, South side)



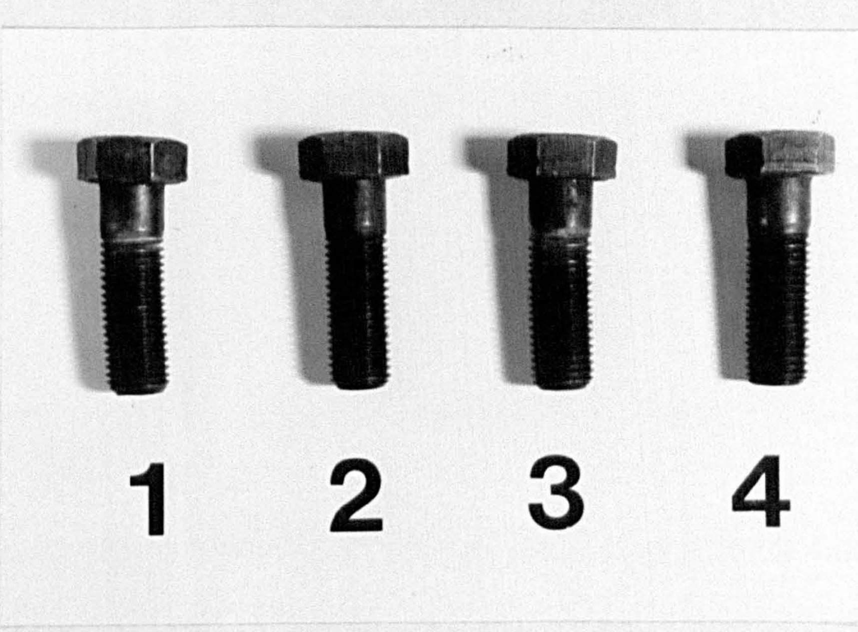
(b) Concrete cone around the stud (Test 4, South side)

Plate 13, Specimen after removal of slab, and concrete cone (Test 4)

Plate 14, Deformation of stud connector (Test 4)



(a) Column section after dismantling (Test 5)



(b) Deformation of bolts (Test 5)

Plate 15, Column after dismantling and deformation of bolts (Test 5)

Chapter 6
ASSESSMENT OF TEST RESULTS

6-1-Introduction

The testing procedure and test observations have been described in Chapter 5. In Chapter 6, the results obtained in the tests will be discussed in detail.

The methods used in the calculation of resistances will first be explained. The calculated resistances are given in Appendix D using the measured dimensions and material properties. The following are given: the resistances of the components of steelwork connection; the moment resistance of the steel and composite connection; the moment resistances of the steel beam and the composite beam. In calculation of the moment resistances of the composite connection and the composite beam in hogging bending, it is assumed firstly that only the rebars are effective and secondly the rebars plus the mesh. For the sagging moment resistance of the composite beam, the mesh reinforcement is ignored. In order that comparison can be made with the test results, no partial safety factor was applied to these calculated values. The summary of the resistance moments is given in Table 6-1.

In later sections of this chapter, the experimental results will be studied, with the experimental values being compared with the calculated resistances. The major and minor axis tests will be treated separately and the influence of parameters varied in the tests will be presented. The experimental behaviour of the connections will be compared with the classification boundaries of joints given in EC3 and EC4. Comparison will also be made between the tests conducted by the author and the similar tests elsewhere. The subject of column stability will be investigated using the results of the minor axis tests.

6-1-1-Resistance Moment of Steel Connection

The tensile resistances of the components of the steelwork connection were calculated using Annex J of EC3. The resulting moment resistance based on the weakest component was then determined assuming that the tensile forces acted at the level of the upper bolts. In general, for an extended end plate connection, assuming R_{b1} as the tensile resistance of the zone between beam flanges, and R_{b2} as the tensile resistance of the extended part, the resistance moment will be:

$$M_{sc}=R_{b1}(D_{b1}-\frac{T}{2})+R_{b2}(D_{b2}-\frac{T}{2}) \tag{6.1}$$

In the case of flush end plate, the second term in Eqn. (6.1) is omitted.

6-1-2-Resistance Moment of Composite Connection

The procedure of calculating the resistance moment of composite connection with a flush end plate has been described in 5-1-2 and Fig. 5-6. For an extended end plate the formula for the resistance moment of the composite connection can be written as:

$$M_{cc}=R_r(D_r-\frac{T}{2})+R_{b1}(D_{b1}-\frac{T}{2})+R_{b2}(D_{b2}-\frac{T}{2})-(R_r+R_{b1}+R_{b2}-R_f)(\frac{x}{2}+\frac{T}{2}) \tag{6.2}$$

It is apparent that this can also be applied to a flush end plate connection by setting R_{b2} to zero. Eqn. (6.2) can also be written in terms of the resistance moment of steel connection:

$$M_{cc}=R_r(D_r-\frac{T}{2})+M_{sc}-(R_r+R_{b1}+R_{b2}-R_f)(\frac{x}{2}+\frac{T}{2}) \tag{6.3}$$

6-1-3-Resistance Moment of Steel Beam

The plastic resistance moment of the steel beam was calculated taking account of the different yield strengths of the web and the flanges. The web and the flange of the beam sections were checked to BS5950:Pt 1 with their measured strengths and found to be Class 1 Plastic. Therefore the beam sections were capable of developing the full plastic moment resistance, with substantial rotation capacity.

6-1-4-Resistance Moment of Composite Beam

The resistance moments and classification of the composite beam sections have been determined in accordance with Appendix B of BS5950:Part 3.1, for both sagging and hogging bending, assuming full shear connection. In this appendix, where the web is in Class 3, an effective depth of web is assumed in compression, and treated as Class 2. The sheeting and tensile strength of concrete were neglected in calculating the negative moment resistance. The mesh reinforcement was ignored in determining the resistance of the composite beam in positive bending.

In calculating positive moment resistance of the composite beam, first the modified compressive strength taking account of the parabolic shape of the stress block was taken from Kong & Evans(1987) as:

$$f_{cu,mod}=0.67\left(1-\frac{\sqrt{f_{cu}}}{52.5}\right)f_{cu} \tag{6.4}$$

Secondly, the value of $0.67f_{cu}$ given in BS5950:Pt 3.1 was assumed for the concrete strength in compression and the results were compared. The former gave the sagging resistance moments about 1% less than the latter. The lesser values of resistance moments are given in Appendix D and Table 6-1.

6-1-5-Predicted Resistance Moment of Connection

The lower of the hogging resistance for the composite beam and the resistance of the composite connection (both including mesh) has been taken as the calculated resistance. This is compared with the experimental value in Table 6-2. In this table, the mode of failure is also given, which in all cases except one was as the predicted mode. The exception was Test 4 in which premature weld fracture interrupted the test. In most cases, first sign of failure was observed in a connection component which was followed by failure of another component. Table 6-3 presents the resistance moments in non-dimensional form, relative to the calculated positive resistance.

6-2-Analysis of Test Results

The moment-rotation curves of all tests have been given in Figs. 5-12 to 5-22. In this chapter, where comparison is made between tests, or with Eurocodes' classification, the moment-rotation curves have been "smoothed out" by excluding the unloading and reloading parts. The variation of strains in the reinforcement with the applied moment is given in Appendix E. In all these curves, the moment is that produced by the applied load at the column face.

The classification of composite beams in hogging bending has been checked according to BS5950:Pt 3.1. The results are tabulated in Table 6-4.

6-2-1-Major Axis Tests

6-2-1-1-Test 1

The moment-rotation curves of Test 1 have been given in Fig. 5-12. In this test, the critical component was the column flange in tension. As the deformation of column flange developed, strain in the reinforcement increased. At the calculated maximum load, the strain gauges on the rebars showed between 9 to 12 $m\varepsilon$, and one of them increased to about 20 $m\varepsilon$ at the next increment. The gauges were found to have passed their strain limit when a further increment was applied. By this stage the column flanges had deformed significantly. Therefore failure was initiated by the column flange which was followed by the fracture of rebars. However, limited flange buckling was observed as the resistance moment of the composite beam was close to that of the connection. Hence, the bar fracture may also be considered as the result of the attainment of the beam's resistance.

Due to the position of the plastic neutral axis, the web was classified as Class 3.

6-2-1-2-Test 2

The moment-rotation curves of Test 2 have been given in Fig. 5-13.

Reference to Table 6-1 shows that the resistance moment of composite connection is slightly greater than that of the composite beam. The failure mode was then predicted to be the beam's bottom flange in compression.

As a much stiffer steelwork connection was utilized, the rotation was modest before buckling of the steel section began. The initial stiffness was twice that of Test 1. Since failure was by local buckling of the steel beam, deformations of the end plate and the column flange were not large and the mesh and the reinforcement did not fracture.

Comparing the moment-rotation curves of Tests 1 and 2 in Fig. 6-1, it is seen that the stiffer steelwork connection does not increase the moment resistance of the composite connection by an amount equal to the difference in the resistance of the steel connections. Comparison between the strains in rebars shows that at loading stages corresponding to the same joint rotations, the strains of Test 2 were much lower than those of Test 1. Therefore, the buckling of the steel beam in Test 2 was not associated with bar fracture. For Test 1, in contrast, the failure of the steelwork connection as a result of the column flange deformation was associated with bar fracture. It can be concluded that the resistance of the steelwork connection did not develop fully in Test 2.

The difference of 30 kNm between observed maximum moments in Tests 1 and 2, compared to the difference of 60 kNm in the calculated moment resistance of their steelwork connection, is due to this fact.

Comparison also shows that the resistance of the specimen in Test 2 decreased as buckling occurred, while in Test 1 the load could be sustained with greater rotation, until the reinforcement fractured. It can therefore be concluded that the use of a full strength connection can improve the moment resistance of the composite joint, but its rotation capacity will be influenced by any tendency to local buckling in the steel beam.

It will be seen later that the amount of reinforcemont does not influence the initial stiffness of the connection significantly. The use of a less reinforced slab with a stiffer connection, such as extended end plate in comparison with flush end plate, may result in the same order of the joint stiffness, but failure may occur in the reinforcement. It is also noteworthy that if the column of Test 1 had been stiffened in tension zone, a behaviour similar to that of Test 2 could have been expected, as the limited deformation of the column flange would not have caused the reinforcement to fracture. However, experimental evidence is needed for these cases.

6-2-1-3-Test 3

The moment-rotation curves of Test 3 have been given in Fig. 5-14. Test 3 is an example of a partial strength and less rigid connection. Due to the lower percentage of reinforcement, it was much weaker than the connection of Test 1, although a similar flush end plate joint was used. Both its moment resistance and rotation capacity were about two-thirds of those of Test 1, limited by the fracture of the mesh and rebars. In this test, only one rebar fractured first, but rotation still increased under almost constant load until a further bar fractured. This can be seen from the moment-rotation curve in Fig. 5-14.

Due to the amount of reinforcement, the composite beam was Class 2 Compact, with the plastic neutral axis in the web. The beams of Tests 1 and 2 were not Compact as their plastic neutral axes were in the upper steel flange.

The maximum possible moment and rotation capacity of a composite connection comprising the flush end plate steelwork joint was not utilized in this test. The additional reinforcement adopted in Test 1 improved the rotation capacity and moment resistance of the connection significantly.

6-2-1-4-Test 4

The moment-rotation curves of Test 4 have been given in Fig. 5-15. In this test, the critical component according to Annex J of EC3 was the column flange in tension. The composite beam was Class 3. The presence of a high amount of reinforcement shifted the plastic neutral axis further up into the upper flange.

The values of resistance moment of composite connection given in Table 6-1 have been found assuming that the shear studs in the hogging region could develop their full characteristic resistance, as recommended by EC4. Therefore the value of R_r in Eqn. (6.2) is the tensile resistance of the bars. If R_r is taken as the maximum shear resistance of 7 stud connectors using BS5950:Part 3.1, with their characteristic resistance value in Grade 40 concrete corrected by the ratio of 0.6/0.8 for negative bending, the moment resistance of composite connection will be 252 kNm. The reduction factor k for connector resistance with profiled steel is 1.0.

After the removal of the concrete around the studs, the 6th and 7th studs were found to have remained straight, while the rest had been bent severely, as shown in Plate 13. This implies that only the remaining 5 studs had been effective. The moment resistance of the composite beam was then calculated assuming that the reinforcement had been carrying a force equal to the shear resistance of a group of 5 studs. This moment is close to the maximum moment achieved in the test.

The concrete locked around the studs was in the form of a cone as shown in Fig. 6-2 and Plate 13. It was crushed in compression where it had been pushed by the stud against the rib of decking. The deformed shape of stud is shown in Plate 14.

The following were observed in the test: uplift of concrete from the steel deck, splitting of concrete along the longitudinal centreline and local failure around the load point. The cause of these could have been due to the following:

a) lack of end anchorage of the reinforcing bars,

b) longitudinal shear failure in the slab associated with the studs,

c) inverted punching failure around the studs due to incompatibility of the stiff slab and flexible steelwork.

Because of the local failure around the load points (which started at 140 kN), the applied load was thereafter acting on the steel beam directly through the part of concrete held under the load point. This case may occur in practice where a column is supported by a steel beam in a composite structure, particularly at the end of a cantilever. The punching failure of a concrete slab subjected to concentrated load has been studied by Johnson & Arnaouti(1980). The shapes of the failure surfaces were similar to those observed in Test 4 at the load points.

The splitting of the concrete slab can be considered as a result of shear failure along the connectors and the lack of transverse flexural stiffness. Considering a composite floor supported by secondary beams, the parts of floor across the secondary beams are in hogging bending, whilst the regions between these zones are in sagging bending. Adequate transverse flexural stiffness is required to resist this transverse bending.

In the case of the isolated composite slab of the tests, the shear lag that existed across the slab width caused a transverse bending in the slab. As the connection deforms, the longitudinal bars placed close to the column are subjected to more tensile force than those away from the column. Their extension is then more, and therefore the length of the edge bars are less than the middle ones. The slab length at the edges tends to remain unchanged as the deflections increase. This causes the slab to bend transversely. The transverse reinforcement would decrease the shear lag effect and prevent the splitting of concrete slab.

It is deduced from the above discussions that the transverse reinforcement shown in Fig. 6-2 can be adopted to improve the behaviour of concrete slab in shear, punching and transverse bending.

The behaviour of the connection in this test was not representative of a highly reinforced flush end plate connection due to the premature fracture of the weld and the later failure of concrete slab. A repeat of this test was therefore undertaken with provisions to prevent any local failure.

6-2-1-5-Test 7

Test 7 was the repeat of Test 4 but with alterations in reinforcement detailing. They were as follows (see Fig. 5-5):

a) end anchorage provided by bends to the end of longitudinal bars,

b) transverse bars with end bends,

c) diagonal bars with hooks.

It is believed that the high amount of longitudinal reinforcement requires an appropriate amount of transverse bars. For the design of reinforced concrete slabs, codes of practice including BS8110(1985) have recommended a minimum amount proportional to the main bars. The minimum amount of transverse reinforcement in composite slabs recommended by BS5950:Part 3.1 is usually satisfied by the mesh reinforcement and the profiled steel sheeting. Improvement in the behaviour of composite slabs with additional transverse bars has been reported by Bernuzzi et al(1991).

The moment-rotation curve of Test 7 has been shown in fig. 5-18. As described earlier, the specimen did not fail prematurely. The specimen sustained the maximum predicted load based on the resistance of the composite beam until a significant rotation capacity was achieved. The mode of failure was not in longitudinal shear, but in local buckling of the compression flange, as predicted by calculation.

Comparing the crack pattern in Test 7 with 1.5% reinforcement, to those in Tests 1 and 3 with 1% and 0.5% reinforcement respectively, it can be concluded that the cracks in a more reinforced slab are more in number than in a less reinforced slab. This can influence the extent of redistribution of moments. However, the sum of crack widths in Tests 1, 3 and 7 under serviceability load (given in Appendix F) increased as the amount of reinforcement decreased.

Large deformation of one or more of the steel components by bending or local buckling, rather than sudden failure of the slab components, is the preferable failure mode because no sudden drop in resistance occurs. The connection of Test 7, like Test 2, is an example of such behaviour. By providing a high amount of reinforcement in Test 7, the behaviour of the extended end plate used in Test 2 was approximately repeated. The maximum moment in Test 2 was slightly higher than in Test 7, but their stiffnesses are comparable.

6-2-1-6-Test 9

The critical component in this bare steel flush end-plate connection was predicted to be the column flange in tension. The second lowest resistance was that of the end-plate in tension. The failure mode in practice confirmed these predictions, by observation of the deformations of the components.

Fig. 5-20 shows the experimental curve of Test 9. Also shown on the plot is the moment-rotation curve obtained by using the method given in Annex J of EC3. It is concluded that this method underestimates the stiffness of the connection, as well as its ultimate resistance. The design resistance obtained by this method is safe and the conservative result obtained might be considered to be due to the idealisations of the calculation models, including the neglect of strain-hardening. The predicted and the test resistances are compared in Table 6-2.

Among the tests on steel major axis joints for which sufficient data is available, the test conducted by Ostrander(1970) is similar to Test 9. The moment-rotation curves of these two tests are compared in Fig. 6-3. It is seen that in the absence of experimental data for a particular connection, results obtained from similar tests can be used.

In Fig. 6-4 the moment-rotation curve of Test 9 is compared with the curves obtained from formulae given by Frye & Morris(1975) and Benterkia(1991). Frye & Morris have given a set of formulae for different types of steel connections including end plate joints to stiffened and unstiffened columns. These have been described in Chapter 3. As shown in Fig. 6-4 (curves "A" and "B"), the formula proposed by Frye and Morris underestimates the connection stiffness and resistance. If the beam depth (suggested by Goverdhan(1984)) is substituted for the distance of extreme bolt rows (as used by Frye and Morris), the result will be curves "C" and "D" shown as "Modified Frye & Morris" in Fig. 6-4. It is seen that even the modified formula for unstiffened joint overestimates the connection resistance.

The connection of Test 9 was stiffened in the compression zone and unstiffened in the tension zone. The calculations given in Appendix A shows that stiffening the compression zone altered the mode of failure from the column web in compression to the column flange in tension. A direct comparison of experimental M-ϕ curve with that of Frye & Morris may not therefore be straightforward. However, for a connection stiffened in compression zone but unstiffened in tension zone, the author suggests that the distance between the top bolt row and the centre of bottom flange be taken as the value of d in the Frye & Morris formula for stiffened joint. Thus, the lever arm of the connection forces will be used. The resulting curve is then that shown in Fig. 6-4 as "Author".

The Formula given by Benterkia is illustrated in Fig.6-5. The resulting curve is shown in Fig. 6-4 as curve "E". This formula underestimates both the stiffness and strength of a connection stiffened in compression zone, but it is very close to the EC3 method for prediction of behaviour of such connection.

The order of rotation capacity thought necessary for the plastic design of semi-continuous frames has been achieved in this test. A rotation of more than 50 mrad could be reached without deterioration of the moment resistance.

In Fig. 6-6, the non-dimensional moment-rotation curve of Test 9 is compared with the boundary defined in EC3 for braced frames. The beam span L_b has been taken as 6.0 m and $M_{Pl.Rd}$ as the plastic moment resistance of steel section with a yield strength of 275 N/mm^2. The connection of Test 9 is clearly semi-rigid and partial strength according to this code.

In almost all tests on the composite connections by the author, the experimental moments are higher than the predicted moments (Table 6-2). Test 4 is an exception in which premature failure occurred due to the weld fracture and local failure. It is seen from Test 9 that the calculation of the resistance of the steel joints to EC3 is a significant underestimate of the experimental value.

Calculations were performed for Test 1, assuming that the steelwork connection attained its maximum experimental resistance of 105 kNm. The corresponding values of R_b and then the depth x (Fig. 5-6) were calculated assuming that R_r was equal to the maximum tensile resistance of the bars. The moment capacity of the composite connection was then calculated as 231 kNm and 246 kNm excluding and including mesh respectively. Comparing these values with experimental value of 262 kNm, it is concluded that the maximum capacity of the steelwork joint had been utilized in Test 1. The rebars had fractured after the attainment of the resistance of the steel connection. The assumption of a stronger steelwork connection increases the value of R_b which in turn will increase the value of x (see Fig. 5-6). When Eqn. (6.2) is used to determine the moment resistance of the composite connection, the increase in the value of x will partly cancel the beneficial effect of additional tensile resistance.

Similar calculations were performed for Test 3 in which the mode of failure was the same as in Test 1. The values of 181 kNm and 209 kNm were then found for the resistance of composite connection excluding and including mesh respectively. These are compared with the experimental value of 179 kNm. It can be concluded that the maximum capacity of the steelwork connection had not been employed in Test 3, and the rebars fractured before the attainment of the resistance of the steel connection.

The amount of reinforcement used in Test 1 proved to be preferable in utilizing the maximum resistance of the steelwork joint.

6-2-1-7-Test 10

The moment-rotation curve of Test 10 has been shown in Fig. 5-21. The weaker components were the column flange in tension and the reinforcement. The predicted maximum load was 292 kN, greater than the shear resistance of the bottom bolt row. Therefore the remaining shear had to be resisted by the top bolt row. The situation at 292 kN was as follows:

vertical load	292 kN
characteristic shear resistance per bolt row	236 kN
shear to be resisted by top bolt row	56 kN
maximum predicted tension per bolt row	263 kN
characteristic tensile resistance of a bolt row	352 kN

The interaction of shear and tension in the top bolt row can be written in accordance with EC3 as:

$$\frac{263}{352} + \frac{56}{236} << 1.4$$

Therefore the characteristic resistance of the bolts was adequate. The friction between the end plate and the column flange was overcome at this stage (292 kN), but not the capacity of the bolts. The maximum compressive force between these plys were about 810 kN, implying a friction coefficient of 0.36. This value can be compared with the slip factors given in EC3. These vary from 0.50 for Class A surfaces to 0.20 for Class D surfaces. The value of 0.36 lies within the slip factor of Class B-C surfaces according to EC3.

A limited rotation capacity was observed in Test 10, being of the order of 14 mrad. The connection was stiffer than other tests using a less deep section but with the same amount of reinforcement, namely Tests 1, 2 and 5. The behaviour of these connections are compared in Fig. 6-1.

The ductility of reinforcing bars met the requirement of BS4449(1988), the elongation at fracture being of the order of 17%. The extension of the bars in Tests 1 and 10 can be calculated at failure by using the position of plastic neutral axis and the rotation of connection at maximum load. These rotations are 28 mrad and 14 mrad which give the extensions of 7.24 mm and 6.30 mm for Tests 1 and 10 respectively. Assuming mesh is also effective, the extension of rebars will be 5.52 mm and 5.71 mm respectively. These values show that although the rotation capacity of Test 10 is half of that of Test 1, fracture of reinforcement occurred at the similar levels of strain (as expected), due to the shift in the position of the neutral axis.

The composite beam was in Class 3 as the web was not compact. It is clear from Test 10 that where the steel beam is deep, a low amount of reinforcement would result in poor ductility of the composite connection. Therefore a criterion for choosing an appropriate reinforcement detail should be based on the cross sectional properties of steel section, and the position of plastic neutral axis of the composite section. This matter will be discussed later in this chapter.

The crack pattern of Test 10 was similar to that of Test 1, except for the number of transverse cracks in the vicinity of the joint, which was 50% more in Test 10. Also a complete longitudinal crack occurred on the centreline of the specimen in Test 10. This implies that the formation of cracks is not dependent only on the amount of reinforcement, but also on the influence of the steelwork components.

Prior to failure, the stiffness of the connection both in loading and unloading, was still close to the initial stiffness of the joint. This is in agreement with the lack of ductility in this test.

6-2-2-Minor Axis Tests

6-2-2-1-Test 5

The moment-rotation curve of Test 5 has been given in Fig. 5-16. In this test, the critical component under balanced loading was predicted to be the end-plate in tension. The calculation was performed using Annex J of EC3 but ignoring failure modes irrelevant to minor axis connections. The resistance moment of the connection was therefore based on the ultimate resistance of the end-plate. The corresponding maximum vertical load applicable to the beam was then used to check the resistance of the bolts in shear and bearing. The shear resistance proved to be sufficient as the bolts were in double shear. The bearing capacity of neither bolts nor the column web was satisfactory. It was assumed however that although ordinary bolts had been used, the friction resulting from the compression forces in the beams could assist in resisting vertical shear.

Following testing and after removal of concrete slab, the column web and the bolts were examined (see Plate 15). No distortion was visible and no change in the hole dimensions was found. Plastic deformation was visible in the bolts from the top row and they were loose when unscrewed. The reason was that in unloading the specimen, the end plates relaxed but the irreversible strain in the bolts remained (see Fig. 6-7(b)).

Comparing $M-\phi$ curves in Fig. 6-1 for Tests 1 and 5, both having the same end plate and reinforcement, it is clear that the connection to the column flange (Test 1) is more flexible than that to the column web (Test 5). The ultimate resistance of the former is also less, as expected. Nevertheless, their rotation capacities are comparable. The stiffness of the connections of Test 5 is similar to the extended end plate connection of Test 2 with the same reinforcement detail, as shown in Fig. 6-1. The reason is that the deformation of column flange was limited in Test 2 by using an extended end plate joint.

6-2-2-2-Test 6

The moment-rotation curve of Test 6 has been given in Fig. 5-17. It is seen that the rotation capacity of the connection is very small due to the brittleness of the mesh reinforcement. In Test 6, after the mesh had fractured, the connection could virtually be considered as a minor axis steel connection in which the critical component would be the end-plate in tension. Although high levels of strain were evident on the end-plate, the cause of failure was stripping of the bolts.

The bolt failure is thought to be as the result of the prying action of the end-plate, since the bolt resistances were much higher than the applied force. The calculated characteristic resistances of one bolt and the connected ply are as follows:

a) bearing resistance:

resistance of bolt	155 kN
resistance of column web	74 kN

b) shear resistance:

resistance of bolt	92 kN

c) tension resistance:

resistance of bolt	176 kN

For comparison, the total vertical load from both jacks was less than 200 kN, assumed to be resisted by the two bottom bolts. The maximum tensile force caused by the maximum moment was 296 kN, shared by the two top bolts. After dismantling the specimen, there was no sign of distress due to bearing, punching or bending on the column web, confirming the adequacy of the bottom bolts. In addition to the capacity of the bolts in shear and bearing, it is recognised that vertical shear could also be resisted by the friction resulting from the compression forces in the beams.

6-2-2-3-Test 8

The moment-rotation curve of Test 8 has been given in Fig. 5-19. In this test, the reinforcement was again the cause of failure. After the test, the top row bolts were loose when unscrewed. As mentioned earlier, the reason was that in unloading the specimen, the end plates relaxed. With column flange deformation not present, due to the minor axis form of the connection, it had been expected that the end-plate would be the critical component of the steel connection. The observed limited local buckling of the beam flange was not expected prior to the actual test as the connection was partial strength.

The connection of Test 8 possessed a very limited rotation capacity. This is due to the lack of ductility in the overall reinforcement (rebars plus mesh).

6-2-2-4-Test 11

The moment-rotation characteristic of Test 11 is plotted in Fig. 5-22. The weaker components in this test were the end plate and the bolts in tension. The stage at which unbalanced loading commenced was intended to be the same level of applied force to the bolts as in Test 5. This was to enable the results of Test 11 to be compared with Test 5. To determine this load, the moment at which the unbalanced loading started in Test 5 was taken. The force in the top bolt row, R_b , was then calculated. The applied load on the specimen of Test 11 was determined as 60 kN if the force in the top bolt row was equal to R_b.

The moment-rotation curve of Test 11 is presented in non-dimensional form in Fig. 6-6. It is compared with the boundary defined in EC3 for braced frames. The initial stiffness of the connection of Test 11 lies in the rigid region of the graph. This connection is partial strength according to this code. Comparison can also be made in Fig. 6-6 between bare steel major and minor axis connections. It is seen that the stiffness of minor axis joint is twice that of major axis one, but their ultimate moment resistance are close. The rotation capacity of the minor axis connection is shown to be much less than the major axis one. This is just as one would expect by removing the flexible column flange and replacing it by a stiff web.

6-2-2-5-General Remarks on Minor Axis Tests

Consider a minor axis connection comprising a single cantilever beam connected to the web of the column section via a flush end plate joint. The idealised yield line pattern of the column web is that shown in Fig. 6-7(a). It is seen that the web panel deforms along the sagging yield lines while is supported along the hogging yield lines.

For the case of cruciform specimens in which the end plate connections are bolted together through the holes in the column web, the situation differs from above. As moments are applied to the connections, the end plates deform as shown in Fig. 6-7(b), stretching the bolts by the force applied via the bolts' head and nut. It is recognised that this action does not involve the column web, and therefore no deformation should be expected from the web. When the applied moments are not equal though, as in unbalanced phase of the minor axis tests, the deformation of end plate and hence the rotation of one side is more, causing the joint to open. Simultaneously, the other joint which is subjected to less moment closes. This action takes place as a result of the force applied to the end plate of the "closing" connection via the bolts head and nut. Again, the column web is not involved in this action and no deformation occurs in the column web. It is concluded that in the unbalanced phase, it is not the flexibility of column web which contributes to the rotation of the connection; rather it is the end plate and bolts that deform and influence the unloading stiffness of the minor axis connection.

As well as stretching of the top bolts and deformation of the end plates, the rotation of the connection includes bending of the column about its minor axis, caused by the force applied at the top of it. The deformed shape of the column is shown in Fig. 6-7(c).

To summarize therefore, the connection rotations measured in the minor axis tests consists of the following:

a) deformation of the end plate;

b) elongation, bending, and at higher loads, stripping of the bolts;

c) bending of the column.

It should be noted that the rotation of the connection of each beam was measured at the level of the top bolts with electronic inclinometers. In retrospect, the reference inclinometer on the column flange should also have been mounted at this level to ensure accurate measurement of the joint rotation. To quantify the resulting error, the free body diagram of the column shown in Fig. 6-7(d) is considered. The bending moment diagram of the column is also shown in the figure. Assuming yield moment at the top bolt row, the moment at the bottom flange of the cantilevers is a quarter of the yield moment. From the curvature along the column to the location of inclinometer, a value of less than 1.0 mrad is calculated for the rotation of the column section. This small value is the maximum possible error that might have occurred in the tests.

When the end plate or bolts do not govern the design of the steel end plate connection, a greater moment resistance is expected from a minor axis connection in comparison with a similar major axis joint. Calculations given in Appendix A show that the failure mode of the major axis joint is that of combined bolts and column flange failure. For the minor axis joint, the failure mode is that of combined bolts and end plate. The maximum moment obtained for the bare steel minor axis joint of Test 11 was 105 kNm which is equal to that of the bare steel major axis connection of Test 9. Thus the resistance of the joints was limited by the bolt resistance. The ductile excessive deformation of column flange in Test 9 did not allow the semi-brittle failure of the bolts.

Whilst in Test 11, the modest deformation of end plate and column web resulted in the stripping of the bolts. The load still could be sustained in Test 9 when it was terminated. Test 11 was terminated because of concern over safety if bolt failure occurred.

6-3-Classification of Composite Connections

The non-dimensional form of moment-rotation characteristic for each composite connection was obtained in accordance with EC4. A cracked beam section was assumed to determine EI_b. The beam span L_b was taken as 9.0 m except for the deeper beam of Test 10 which was taken as 11.5 m. The plastic moment resistance was calculated for the composite beam in hogging bending, excluding mesh, but assuming full shear connection. The measured values of dimensions and strengths of flanges and web of steel sections were used to determine $M_{Pl.Rd}$. The hogging resistance moments have been given in Table 6-1. The experimental moments were those produced by the applied load at the face of the column.

According to EC4, the classification as rigid or semi-rigid depends on the value taken for the flexural rigidity of the connected beam (cracked or uncracked); also using different lengths for the adjacent beams can change the classification. However the classification of connections by moment resistance is independent of the beam length.

Comparison will be made in this section for the major and minor axis connections separately. All tested connections then will be compared together. In each case, the behaviour of connections will be compared with the boundaries for rigid connection defined in EC4 for braced frames.

6-3-1-Major Axis Connections

The moment-rotation curves of the six major axis composite connections are compared in Fig. 6-8.

As shown, the stiffness of connection of Test 10 that used the deeper beam is greater than the other flush end plate connections. The rotation capacity of the connections of Tests 1,2 and 7 are comparable. A minimum value of 0.40 for $\bar{\phi}$ has been achieved in these tests. In Test 4, similar rotation capacity is shown on the plot, but with deterioration of the moment. Due to the weld fracture in Test 4, the behaviour of this connection should be excluded from the rotational characteristics of these tests. The improved behaviour of Test 7 owes its performance to the provisions of the reinforcing details. The high value of rotation capacity observed in Test 7 was more than is thought to be required of a composite connection in semi-continuous framing.

Comparing the behaviour of the connection in Test 7, in which 1.5% reinforcement was used, with that of Test 1 comprising 1% reinforcement, the former appears to be over-reinforced without achieving much higher moment resistance and stiffness. Regarding serviceability state, their stiffness at the two-thirds of their resistance is also close.

The connections of Tests 1, 2 and 7 are full strength compared to the moment resistance of the composite beam as shown in Fig. 6-8. The connections of Tests 3 and 10 are partial strength. All the tested major axis connections except Test 3 are classified as rigid to EC4. It is difficult to classify the connection in Test 3 according to this code. The connection in Test 3 behaved initially rigid, then semi-rigid as it crossed the boundary line of EC4. Since the moment corresponding to this intersection point is less than two-thirds of the adjacent beam, it can be considered as semi-rigid.

Generally, the stiffness of reinforced major axis connections increases slightly with more reinforcement, but the increase is not to the same extent as the moment resistance.

6-3-2-Minor Axis Connections

The moment-rotation curves of the three minor axis composite connections are compared in Fig. 6-9. Shown in Fig. 6-9(a) is the behaviour of the connections on the side that was unloaded. The unloading part of the $M-\phi$ curve is omitted and the curve is assumed continuous. Fig. 6-9(b) shows the behaviour of the connections for the side on which load was maintained constant in the unbalanced phase. The part of the $M-\phi$ curve in which the moment was constant is omitted. These changes are made to enable the $\bar{m}-\bar{\phi}$ curves to be used for connections without unbalanced loading, in particular for classification.

The stiffnesss of the connections in both Fig. 6-9(a) and (b) are close. Generally, the amount of reinforcement does not seem to be of much influence as far as the stiffness of a minor axis connection is concerned. This is because the web is much stiffer than an unstiffened column flange, and is determining in the rotational behaviour of the composite joint.

The rotation capacity of connections in Tests 6 and 8 is limited. This is due to the lack of ductility in the overall reinforcement (rebars plus mesh) of low-reinforced connections. However the connection of Test 5 possessed a higher rotation capacity; it was more substantially reinforced than Tests 6 and 8.

The moment-rotation characteristics of the minor axis connections are compared in Fig. 6-9 with the EC4 boundary for rigid connections. As shown, all tested flush end-plate connections bolted to the web of the column are classified as rigid to EC4. The connection of Test 5 is full strength and those of Tests 6 and 8 are partial strength. The composite beam in Tests 6 and 8 is Class 2 Compact as the plastic neutral axis lay in the web of the steel section, well below the top flange.

6-3-3-Comparison between Major and Minor Axis Connections

Fig. 6-10 gives a comparison between the behaviour of all the composite connections tested by the author. The minor axis connections are clearly stiffer than major axis ones. In general, the rotation capacity of the former is limited due to the rigidity of the steelwork joint. The amount of reinforcement appears to increase the resistance moment of the connection, but this beneficial effect is mostly for the amounts up to 1%. It can be concluded that when the reinforcement area exceeds an optimum value, the joint ultimate resistance and stiffness do not increase significantly.

6-4-Effect of Variables on Behaviour of Connections

Different joint behaviour is expected if the configuration of steelwork components and the detailing of concrete slab vary. Also the behaviour of test specimens differ if the loading type and sequence change.

The types of failure of composite joints can be divided into two categories:

a) Primary failures; such as failure of the components of the steelwork connection, fracture of reinforcement, and local buckling of the steel beam adjacent to the joint.

b) Secondary failures; such as failure of the shear connector in bending, fracture of the connector's weld, failure of concrete around the connector, failure in vertical shear, and crushing of concrete against the column.

In order to prevent the premature primary failures, adequate design of the steelwork joint and appropriate slab detailing must be adopted. The secondary failures will be prevented if the joint is capable of resisting vertical shear, full shear connection is provided and the transverse reinforcement is adequate.

6-4-1-Reinforcement Detailing and Depth of Beam Section

An appropriate amount of reinforcement is necessary to provide the degree of continuity required in a semi-continuous design. Furthermore, the ductility of reinforcement must be such that the connection can rotate without deterioration of its moment resistance to allow the degree of redistribution assumed in design to take place. Care should be taken to prevent brittle behaviour of the overall reinforcement.

The ultimate resistance of the stiffened composite connection depends mainly on the interrelation of reinforcement in tension, and the susceptibility of the steel section to local buckling in compression. Therefore the reinforcing details must be determined in conjunction with the slenderness of the component parts of the steel section.

The first attempt to provide an indication of the appropriate reinforcement, was to relate the tensile resistance of the reinforcement to the tensile resistance of the steel section. Johnson & Hope-Gill(1972) suggested a non-dimensional "force ratio":

$$\Phi = \frac{A_r f_{yr}}{A_s f_{ys}} \tag{6.10}$$

Force ratios from 0.16 to 0.44 were used in their research. The force ratios in the tests conducted by the author are given in Table 6-5. It varies from 0.15 to 0.45 if mesh reinforcement is excluded, or from 0.22 to 0.52 if mesh is included.

The rotational behaviour of Tests 3 and 10 was poor, both possessing small force ratios. The greatest force ratio was with Test 7 (1.5% reinforcement). The result was to increase the rotation capacity of the connection. The required rotation of connections in semi-continuous framing will be discussed in Chapter 8. However, the rotation capacity required for plastic design is in the order of 15-30 mrad for the cases considered in Chapter 8.

The behaviour of tests with 1% reinforcement proved to be satisfactory regarding both the ultimate moment and rotation capacity. This amount of reinforcement corresponds to a force ratio of 0.30, or if mesh is taken into account, 0.37. It can therefore be concluded that a force ratio between 0.3 and 0.4 would be preferable to achieve both economy and the behaviour required for design of semi-rigid frames. A force ratio of 0.5 corresponding to the highly reinforced connection can be considered as an upper limit to the reinforcement, since the moment resistance of the joint is limited by the local buckling resistance of beam.

A "semi-rigid force ratio", ρ_F, was proposed by Zandonini(1989) as the ratio of the resistance of reinforcing bars to the resistance of the bottom beam flange:

$$\rho_F = \frac{A_r f_{yr}}{A_f f_{ys}} \qquad (6.11)$$

This ratio was supposed to be appropriate to control local buckling in design because of the reinforcement force being related to the resistance of the compression flange. It was suggested to be limited to 1.0. This ratio varied from 0.48 to 1.38 in the tests by Johnson & Hope-Gill(1972). Zandonini(1989) has reported that the researchers considered have used a ratio from 0.27 to 1.38. Zandonini thought that this ratio was rather conservative, when he examined the results by Johnson & Hope-Gill. Hence he suggested that the values greater than 1.0 might be acceptable when the steel flange is Compact.

The "semi-rigid force ratio" is given for the tests conducted by the author in Table 6-5. Although the ratio of 1.0 corresponds to the tests with 1% reinforcement, it does not distinguish between Test 10, in which a poor rotation capacity was observed, and the well behaved tests. It does not differentiate between two steel sections of different depths but with similar area of bottom flange.

The classification of composite section in terms of slenderness of parts in compression is an influential parameter in determining the appropriate amount of connection reinforcement. The more reinforcement is used, the higher the neutral axis and consequently the greater depth of web in compression. This results in a more slender web which increases the susceptibility of the steel section to local buckling. On the other hand, if a low amount of reinforcement is used, the rotation capacity needed for moment redistribution will not be achieved because of fracture of the reinforcement.

The plastic design of composite beams requires sufficient support rotation to take place for a plastic mechanism to develop. In order to determine the moment resistance of a composite beam by plastic methods, the composite section must be either in Class 1 Plastic or Class 2 Compact. This necessitates the web of the steel section to be in Class 1 or 2. It is realised that the slenderness requirement of the web limits the amount of reinforcement, while the rotation requirement demands more reinforcement.

The preceding discussions suggest that an optimum value exists for the area of reinforcement over the supports of composite beams jointed in a composite frame. In the force ratios suggested by Johnson & Hope-Gill and Zandonini two parameters have been taken into account:

1) the total resistance of reinforcement,

2) the resistance of the steel section or its flange.

The "force ratio" relates the cross sectional properties of the steel beam to the reinforcement detailing by including the area of the steel section. The "semi-rigid force ratio" has the advantage of taking account of the effect of local buckling of the compression zone of the beam section by including the strength of the bottom steel flange. A greater resistance to the bottom flange results in a composite section which is less susceptible to local buckling, because of the strength provided for the compression zone and its effect on the position of plastic neutral axis.

The "semi-rigid force ratio" still needs a measure of the depth of the steel section as well as an indication of the relative strengths of tension and compression zones of the composite section. The latter can be allowed for by the position of plastic neutral axis.

To summarize, the following parameters are proposed to be taken into account, in conjunction with the "semi-rigid force ratio":

1) the position of plastic neutral axis,

2) the depth of the steel beam.

For design, a rather simple rule is needed for determining the support reinforcement. The present author introduces the ratio of:

$$\omega = \sqrt{\frac{x_p}{D/2}} \tag{6.12}$$

where x_p is the distance between the plastic neutral axes of composite and steel sections, and D is the depth of the steel beam.

The "semi-rigid force ratio" has to be multiplied by ω:

$$\rho_r = \omega \cdot \rho_F \tag{6.13}$$

and the resultant has to be compared with 1.0. If it is less or greater than 1.0, the reinforcement detailing may be altered accordingly to allow for full use of reinforcement and steel section. When it is less than 1.0, the reinforcement amount should be increased to approach a value of 1.0 for ρ_r and vice versa. This is shown in the following instruction:

$\rho_r < 1.0$ increase the amount of reinforcement.

$\rho_r > 1.0$ decrease the amount of reinforcement.

This ratio has been applied to the "semi-rigid force ratio" of the tests conducted by the author and the results are given in the last two columns of Table 6-5. Tests 1 and 10 can now be compared together. For Test 1, an average value of 1.0 between the two last columns of the table (related to rebars only and rebars plus mesh) has been found, in contrast with the average value of 0.7 for Test 10. This means that the amount of reinforcement should be increased in the latter, if full use of the connection components is sought. With the same slab width as the author's tests, an area of 1200 mm^2 of reinforcement makes this ratio equal to 1.0. Experimental evidence is yet needed to justify the applicability of the proposed ratio. For Test 7, it has been argued that only a modest improvement in the joint resistance was achieved although a high amount of reinforcement had been used, and the large rotation capacity achieved was not required for design. Accordingly, the average of 1.5 of the last two columns of Table 6-5 can be reduced to 1.0 by simply adopting the 1% reinforcement. This has already been proved in Test 1.

6-4-2-Type of Steelwork Connection

Tests 1, 2 and 5 comprised the same amount of reinforcement and size of steel section. The difference between these connections was the type of steelwork joint. Test 1 utilized flush end plate connections to the column flanges, Test 2 comprised extended end plate joints to the column flanges, and Test 5 employed flush end plate connections to the column web.

The initial stiffness of end plate composite joints is not strongly influenced by the amount of reinforcement, but is much more dependent on the type of steelwork connection. This can clearly be seen from Fig. 6-1, where all three connections have the same amount of reinforcement but different steelwork details. Tests 2 and 5 possess similar stiffness because of little or no deformation of column flange. This deformation was the determining feature in Test 1.

The ultimate behaviour of end plate connections is also dependent on the presence or absence of local buckling in the steel section. This in turn depends on the position of the plastic neutral axis. This is relevant when the connection is full strength, as in Tests 2 and 5.

6-4-3-Type of Loading

The behaviour of the joints of either side of column is not completely independent due to the continuity of the slab. Benussi et al(1987) pointed out that this interrelation increases with the flexibility of the steelwork connection. For the steel joints used in the tests by the author, which are regarded as the upper bound to semi-rigid connections, this interrelation should have a little influence.

The loading sequence has an effect on the results. The method used in these tests was as following: after the specimen relaxed at the previous load level, the load increment was applied. If subsequently there was not significant drop of the load, the results was printed. Otherwise, the stage of loading was restored until the dropping of the load became insignificant. The delay between the application of the increment of load and the printout of results was usually 5 to 15 minutes. It was lesser in the beginning of each test.

However, the maximum load has been taken as the greatest recorded by continuous printing of results during the test. Failure was characterised by an inability to restore the loading after the peak value was applied.

The influence of loading sequence on the stress state in the slab has also been reported by Law(1983). The resulting strain hardening in the rebars is thought to affect the joint moment capacity(Zandonini(1989)).

In the minor axis tests, the stage at which unbalanced loading started was not unique. The determination of this stage was based on the observations of the composite joint and concrete slab, together with the calculated resistance. It was desired to load the specimens to the onset of failure before unloading one side, but care had to be taken to avoid loading so far that the mesh fractured. Therefore the commencement of the unbalanced phase was based partly on judgement.

The ultimate resistance of composite connections in the minor axis tests was also influenced by the degree of yielding of the joint components in the unbalanced phase. This is concluded from the moment-rotation curves of these tests in Figs. 5-16, 17 and 19. In Tests 6 and 8 the final applied moment is more than that at which unbalanced phase started, but in Test 5 the final applied moment is less than at the beginning of unbalanced phase. This implies that the connection components of Test 5 underwent a significant yielding in the unbalanced phase, but in Tests 6 and 8 the yielding did not develop in the same extent as Test 5, to avoid the brittle fracture of overall reinforcement. The maximum moment of the minor axis connections might have been more if increasing load was applied from the start to the end of test, without yielding of reinforcement during the unbalanced phase.

6-4-4-Shear Connection

Both BS5950:Part 3.1 and EC4 require that full shear connection be provided in the negative moment regions of continuous beams. The shear connectors must be capable of withstanding the design tensile resistance of the effective reinforcement. In BS5950:Part 3.1, the design resistance of shear connectors in negative moment regions is taken as 60% of the characteristic resistance, compared to 80% for positive moment regions. The additional reduction in negative bending is due to the influence of cracking of concrete. EC4 does not distinguish between these two cases.

Whether the shear connection in the tests with the highest amount of reinforcement, namely Tests 4 and 7, was partial or full depends on the code. Tests 4 and 7 were partial shear connection according to BS5950:Part 3.1, and full shear connection in accordance with EC4.

It has been shown that in the case of Test 4, only five studs of a total of 7 were effective, leading to longitudinal shear failure in the slab. In Test 7, the mode of failure was not the failure of shear connectors.

The degree of interaction realized by the shear connection is clearly influential in the composite action. The effect of connectors layout has been studied in the tests by Law(1983), but no indication of partial or full interaction behaviour has been given.

The effect of connector type has been examined by Puhali et al(1989). Uplift of concrete slab with respect to the steel beam was experienced where non-welded connectors without connector head were used. They believed that this had not influenced the connection response.

Other than the above studies, no research is known to the author to clarify the influence of shear connectors on the connection behaviour. It will be shown in Chapter 7 that the effect of connectors' flexibility cannot be neglected in the prediction of connection behaviour.

In order to identify the effect of the degree of shear connection in the hogging region adjacent to the composite connections, further experimental and analytical evidence is needed.

Since the main concern in the connection tests conducted by the author was to eliminate any secondary failure such as failure of studs, the test programme was not aimed at investigating the influence of shear connectors.

6-5-Tests on Composite End Plate Connections Elsewhere

A number of tests have recently been conducted on end plate connections in Italy by Bernuzzi et al(1991). More recently, a number of end plate joints have been tested by Xiao et al(1992) at Nottingham University. Table 6-6 summarizes the details of these tests. Fig. 6-11 gives their moment-rotation curves.

6-5-1-Tests in Italy

Three specimens are considered. Specimens SJB10 and SJB14 utilized flush end plates with two rows of bolts, a column web stiffened in the compression zone and a solid slab. Specimen CT2 comprised extended end plates (extended in the compression zone) with three rows of bolts, an unstiffened column and a composite slab. The percentage of reinforcement was 0.7% in specimen SJB10, 1.2% in SJB14 and 1.1% in CT2. The end plates were 12 mm thick, connected to the column flange. Beam sections were IPE300 for SJB10 and SJB14, IPE330 for CT2. The column section was always an HEB260.

6-5-2-Tests in Nottingham

Three specimens comprising flush end plate connections are considered. In specimen SCJ3 only mesh reinforcement was used. Specimens SCJ4 and SCJ5 employed 1.2% reinforcement. SCJ3 and SCJ4 were connections to the flanges of an unstiffened column, whilst the column of SCJ5 was stiffened in the compression zone. All tests utilised profiled steel sheeting and 10 mm end plates with four rows of bolts. Beams and columns were of 305x165UB40 and 203x203UC52 sections respectively.

6-5-3-Comparison between Tests in Italy and Nottingham

Despite the differences in the configuration of connections, the flush end plate composite joints of these series have similarities in behaviour. The overall shape of their moment-rotation curves are similar and their rotation capacity is in the range of 20-25 mrad.

As the steel sections, end plate thicknesses and number of bolt rows were different, it is not possible to compare them in a straightforward manner. However, the beam sections and the percentage of reinforcement were similar in some of the specimens.

Specimens SJB14 and SCJ5 comprised the same percentage of reinforcement and their behaviours are similar. The greater thickness of column flange and end plate in the former made it stiffer. However, the use of a solid slab in the former would not influence the connection behaviour significantly.

The behaviour of specimens SJB10 and SCJ4 (the former stiffened but with less amount of reinforcement) are also comparable. The greater thickness of column flange and end plate in the former has compensated for the greater amount of reinforcement in the latter, regarding both stiffness and resistance.

6-5-4-Comparison between Tests in Warwick, Nottingham and Italy

The moment-rotation curves of these series of tests are shown in Fig. 6-12. The stiffness and resistance of the specimens of Test 1 and SJB14 are similar. The specimens possess the same percentage of reinforcement. Their differences are the thicker end plate in the former and the thicker column flange in the latter. As these differences result in similar moment-rotation curves, this implies that the moment resistance of composite connection is more dependent on the amount of reinforcement. It is seen from Eqn. (6.3) that the term corresponding to the force in reinforcement is dominant where sufficient amount of reinforcement is used, compared to the limited possible change in the moment resistance of flush end plate steel joint.

The specimen of Test 1 is also similar to that of SCJ4, but for the unstiffened column, thinner end plate and more bolt rows in the latter. The thickness of the end plate in SCJ4 is less than that of the column flange and therefore the mode of failure changes from column flange to end plate. The lesser moment resistance of SCJ4 is due to the unstiffened column in compression and the flexibility of end plate, which in turn caused earlier plastification of the reinforcement.

The stiffness and rotation capacity of Test 10 are similar to those of CT2. This implies that higher stiffness can be expected with a thinner end plate and an unstiffened column if the beam section is deeper or end plate is extended below the compression flange of the steel section.

The stiffness of the connection of Test 2 lies between those of CT2 and SJB14, because of the thicker column flange in CT2 and the thinner end plate in SJB14. The extension of the end plate of Test 2 in the tension zone eliminated the failure of column flange. The mode of failure then was beam buckling, which is also the failure mode of SJB14.

Test 3 can be compared with SJB10 since both comprised the same percentage of reinforcement. The latter is stiffer due mainly to the thickness of column flange and partly to the solid slab. The ribs of decking in composite slabs have been found to induce cracks in the tests by the author, which can influence the rotational behaviour of the joint. However the stiffnesses obtained in Test 3 and SJB10 are the least in these series.

The rotation capacities of Tests 2 and 3 are of the same order as similar tests in Nottingham and Italy. The rotation capacity of Test 1 is clearly greater (35 mrad compared to 25 mrad).

It is noteworthy that the connection with only mesh, namely SCJ3, does not possess the least initial stiffness. It is deduced once again that more reinforcement does not necessarily increase the initial stiffness of the composite joint, but the overall ductility of reinforcement is a significant factor in ultimate behaviour.

6-6-Column Stability in Semi-Continuous Framing

A beam-to-column assembly as a part of a braced frame is shown in Fig. 6-13(a). For balanced conditions, no first-order moment exists at the column ends. Assuming unbalanced loading, the first and second order moments at low levels of loading are shown in Fig. 6-13(b). The corresponding deflected shape is shown in Fig. 6-13(c). By increasing the load to the column, this member tends to buckle and stiffness is lost. The beams then tend to restrain the column against further rotation. The degree of restraint provided by the beams depends on the stiffness of these members and the connections. A situation may be achieved in which the moment in a beam will be entirely positive. The bending moment diagram is then that shown in Fig. 6-13(d) and the corresponding deflected shape as Fig. 6-13(e).

The main purpose in applying unbalanced loading in the minor axis tests was to simulate chequer-board loading of a real frame and investigate the degree of restraint provided by semi-rigid connections to a column in terms of effective slenderness.

The extreme situation of entirely positive moment on one side of the column (Fig. 6-13(d)) was not examined in the minor axis tests. This case is very unlikely to happen in braced frames, due to the limited difference in the imposed loads of spans, unless the column is very near to collapse due to buckling. However it is likely in unbraced frames subjected to wind loads, as illustrated in Fig. 4-7. The bending moment diagrams of an unbraced multistorey frame under gravity and wind loads are shown in this figure.

Although connections in braced frames were intended to be tested in the project, the arrangement used did not comply with the terms of a braced frame. A test rig in which the top of the column is held in position is more relevant to a braced frame.

In the case of an external minor axis joint, the flexibility of the column web significantly affects the connection behaviour. The loading and unloading stiffness of such connections includes the stiffness of the column web. This situation is shown in Fig. 6-14. In the case of internal joints, the influence of the stiffness of column web has been hard to recognize in the author's tests.

In the unbalanced phase, the unloading stiffness did not follow the same slope shown by unloading in the balanced phase. It was less steep, as shown in Figs. 5-16, 17 and 19. By unloading one beam, the column web bent and the connection at the side of unloaded beam tended to close. This reduced the connection rotation more rapidly compared to the situation in which both beams are unloaded together.

6-6-1-Quantifying Effective Length of Column

Consider the assemblage of Fig. 6-15(a) in which a column is restrained by two beams simply supported at the ends. The increase in combined dead and live loads will result in plastic hinges forming at the partial strength connections. This situation is shown in Fig. 6-15(b). No rotational restraint is then provided by the beams to the column.

If however the live load decreases on one side, or if reversal of end moment occurs in one beam due to the tendency of the column to fail (see Fig. 6-13), some rotational restraint will be provided by the unloading stiffness of the connection (joint B in Fig. 6-

15(c)). This corresponds to the tangent stiffness of the unloading $M-\phi$ curve in the minor axis tests. Since the $M-\phi$ curve of the unloading connection in the unbalanced phase is approximately linear, as seen in Figs. 5-16, 17 and 19, the tangent stiffness for the ranges of moment in the tests is approximately constant for a particular connection. This is shown by stiffness C in Fig. 6-15(c).

To quantify the effect of semi-rigid composite joints on the column stiffness, the assemblage given in BS5950:Pt 1 and EC3 (Fig. 6-16(a)) is considered. For the situation explained in the last paragraph, the restraint provided by the connections and beams on one side is assumed neglected, and the assemblage will be that of Fig. 6-17. The column and beam sections used in the minor axis tests can be assumed with the column height and beam span as given in Fig. 6-17.

The equivalent beam method described in Chapter 3 can be employed to find the relative stiffness of a beam with semi-rigid connections. Eqn. (3.20) is used for a beam in a sway frame. An expression can be derived from Eqn. (3.19) for a beam in a non-sway mode, assuming $K_A = K_B$:

$$K_r = [\frac{3}{\frac{6EI}{CL}+3}]\,\frac{I}{L} \tag{6.14}$$

For an uncracked composite section, the value of $\frac{I}{L}$ is found from Appendix B of BS5950:Pt 3.1. The values of $\overline{K_r}$ (ratio of relative stiffness of semi-rigid beam to the rigid beam stiffness) are tabulated in Table 6-7 for various values of C taken as the unloading stiffness of the minor axis connections.

In order to find the effective length ratios of the column in the assemblage of Fig. 6-16, the charts given in Appendix E of EC3 (mentioned in Chapter 2) can be used. The values of η_1 and η_2 are equal since the structural elements of the top and bottom storeys are assumed identical. Therefore η can be calculated from the relationship given in Fig. 6-16(a). The values of effective length ratios can then be found for both sway and non-sway modes from the EC3 charts of Figs. 2-12(b) and 6-16(b). The calculated values of η

and the resulting effective length ratios are given in Table 6-7.

As seen from Table 6-7, semi-rigid composite connections have a significant influence on the effective length of columns. The values of unloading stiffness in the author's tests increases from Test 5 to Test 8 and Test 6. This is mainly due to the stage chosen to commence the unbalanced phase as described in 6-2-2-5. The unloading stiffness of Tests 8 and 6 are respectively 2.5 and 3.5 times that of Test 5. However, as the unloading stiffness increases, K_r increases while η decreases which results in a lower value of effective length ratio. The influence of the unloading stiffness on the effective length of columns is more in a sway frame compared to a non-sway frame.

As assuming cracked section throughout the beam length gives conservative results for column restraint, it was examined with the values of second moment of area found from Appendix B of BS5950:Pt 3.1. Only main reinforcement was taken into account. The results are tabulated in Table 6-8.

The results for the bare steel connection of Test 11 are given in Table 6-9. Since in Test 11 two phases of unbalanced loading were carried out, two set of values are given. It is seen from Tables 6-8 and 6-9 that, again, the value of non-sway l/L is not very sensitive to change in the value of C.

6-7-Conclusions

The following conclusions are deduced from the assessment of the test results:

1) The formulae given for prediction of the moment resistance of composite connections are in good agreement with the experimental results. The difference is between -4% and +18%, i.e. the formulae give generally safe values of moment resistance of composite connections. Negative value means that predicted moment resistance was less than test moment.

2) The formulae given by EC3 for prediction of the moment resistance of steel end plate connections underestimate the connection resistance and stiffness. For particular cases of major and minor axis joints tested by the author, the observed ultimate resistance moments are respectively 64% and 42% more than the values predicted by the EC3 method.

3) The modes of failure predicted by using the formulae mentioned in 1 and 2 above were correct in all cases.

4) The main variables affecting the behaviour of end plate connections were studied. These are the amount of reinforcement, the depth of steel beam, the type of loading, the type of end plate connection (i.e. flush or extended end plate and major or minor axis).

5) The amount of reinforcement increases directly the moment resistance of composite connections. Full strength connections can be provided by increasing the amount of reinforcement. The ductility of the overall reinforcement (rebars plus mesh) has a significant influence on the rotation capacity of composite connections. The beneficial effects of reinforcement is most significant for amounts about 1% of the concrete area above decking. The amount of reinforcement does not influence the initial stiffness of composite connections.

6) The increase in the depth of the steel beam will increase the stiffness and decrease the rotation capacity of a composite connection compared to a connection with similar steelwork joint and a similar concrete slab having the same amount of reinforcement.

7) The type and sequence of loading have some effects on the test results. The stage at which unbalanced loading of a cruciform specimen commences has a significant influence on the unloading stiffness of the connection.

8) An extended end plate connection increases the moment resistance of both steel and composite connections compared to similar flush end plate joints. The use of the former decreases the need for stiffening the tension zone of major axis connections.

9) Minor axis connections (both steel and composite) are stiffer than major axis connections with similar connection components. The rotation capacity of the former is limited due to the rigidity of the steelwork joint.

10) A method is proposed to determine the appropriate amount of reinforcement for an efficient design of composite joints. A ratio "ω" is defined to multiply "semi-rigid force ratio". The resultant is compared with 1.0. The assumed amount of reinforcement is increased if the resulting value is less than 1.0 and vice versa.

11) The classification of steel connections given by EC3 results in the tested major axis joint being semi-rigid and partial strength. The tested minor axis joint may be classified as rigid but still partial strength.

12) The classification of composite connections given by EC4 results in all tested composite connections being rigid. The connections with 1% or more reinforcement are full strength while those with less than 1% reinforcement are partial strength.

13) The results of the author's tests are comparable with the experimental results of tests on composite end plate joints elsewhere. The reasons for similarities and differences have been pointed out.

14) For steel flush end plate connections stiffened only in the compression zone, it is proposed to use the prediction equation of Frye & Morris(1975) for stiffened connections with the value of d taken as the lever arm between the tensile force in the top bolt row and the compressive force in the bottom flange of the beam.

15) The effect of semi-rigid connections on column stability has been studied. The results of the minor axis tests have been used in determining the effective length of the column. The change in unloading stiffness was found to be of little influence on

the buckling length of columns. The conventional value of 0.7 taken as the effective length ratio of columns in non-sway frames is reasonable but not necessarily conservative. A value of 0.75 would though be safe.

Table 6-1, Summary of resistance moments

TEST No.	STEEL BEAM	STEEL CONN.	COMPOSITE BEAM		COMPOSITE CONN.		COMPOSITE BEAM (sagging)
			R	R+M	R	R+M	
1	173	65	248	259	219	243	371
2	173	123	248	259	248	263	373
3	174	64	221	239	150	187	377
4	174	64	272	282	267	280	377
5	176	74	251	263	225	248	382
6	175	74	175	200	74	117	380
7	173	63	271	281	264	277	376
8	168	74	215	233	157	192	365
9	173	64	-	-	-	-	-
10	339	102	451	457	328	375	702
11	172	74	-	-	-	-	-

R: Rebars only.
R+M: Rebars plus Mesh.

Table 6-2, Comparison between predicted and test resistance

TEST No.	RESISTANCE MOMENT		$\frac{TEST}{PREDICTED}$	FAILURE MODE	
	PREDICTED	TEST		PREDICTED	TEST
1	243	262	1.08	column flange	column flange followed by reinforcement
2	259	291	1.12	beam flange	beam flange
3	187	179	0.96	column flange	column flange followed by reinforcement
4	280	243	0.87	column flange	local failure of slab followed by shear connection
5	248	293	1.18	end plate and reinforcement	end plate followed by reinforcement
6	117	138	1.18	end plate and mesh	end plate followed by mesh and bolts
7	277	302	1.09	column flange and beam flange	beam flange
8	192	211	1.10	end plate and reinforcement	end plate followed by reinforcement
9	64	105	1.64	column flange	column flange
10	375	416	1.11	column flange and reinforcement	column flange followed by reinforcement
11	74	105	1.42	end plate and bolts	end plate followed by bolts

Table 6-3, Non-dimensional resistance relative to M_{sag}

TEST No.	STEEL BEAM	STEEL CONN.	COMPOSITE BEAM		COMPOSITE CONN.		TEST
			R	R+M	R	R+M	
1	0.47	0.18	0.67	0.70	0.59	0.65	0.71
2	0.46	0.33	0.66	0.69	0.66	0.70	0.78
3	0.46	0.17	0.59	0.63	0.40	0.50	0.47
4	0.46	0.17	0.72	0.75	0.71	0.74	0.64
5	0.46	0.19	0.66	0.69	0.59	0.65	0.77
6	0.46	0.19	0.46	0.53	0.19	0.31	0.36
7	0.46	0.17	0.72	0.75	0.70	0.74	0.80
8	0.46	0.20	0.59	0.64	0.43	0.53	0.58
10	0.48	0.15	0.64	0.65	0.47	0.53	0.59

R: Rebars only

R+M: Rebars plus Mesh

Table 6-4, Classification of beam sections in hogging bending

TEST No.	POSITION OF NEUTRAL AXIS	CLASS OF SECTION
1	TOP FLANGE	3 SEMI-COMPACT
2	TOP FLANGE	3 SEMI-COMPACT
3	WEB	2 COMPACT
4	TOP FLANGE	3 SEMI-COMPACT
5	TOP FLANGE	3 SEMI-COMPACT
6	WEB	2 COMPACT
7	TOP FLANGE	3 SEMI-COMPACT
8	WEB	2 COMPACT
9	WEB	1 PLASTIC
10	WEB	3 SEMI-COMPACT
11	WEB	1 PLASTIC

Table 6-5, Force ratios of the test specimens

TEST No.	FORCE RATIO		SEMI-RIGID FORCE RATIO		AUTHOR	
	R	R+M	R	R+M	R	R+M
1	0.30	0.37	0.96	1.19	0.90	1.15
2	0.30	0.37	0.96	1.19	0.90	1.15
3	0.15	0.22	0.48	0.71	0.30	0.57
4	0.45	0.52	1.44	1.67	1.40	1.63
5	0.30	0.37	0.96	1.19	0.90	1.15
6	-	0.07	-	0.23	-	0.11
7	0.45	0.52	1.44	1.67	1.40	1.63
8	0.15	0.22	0.48	0.71	0.30	0.57
10	0.20	0.26	0.90	1.12	0.61	0.85

R: Rebars only.
R+M: Rebars plus Mesh.

Table 6-6, Details of tests in Italy and Nottingham

SPECIMEN	BEAM SECTION	COLUMN SECTION	END PLATE THICKNESS	NO. OF BOLT ROWS	COLUMN STIFFENER	SLAB TYPE	REINF. %	FAILURE MODE
SJB10	IPE300	HEB260	12	2	yes	solid	0.7	A
SJB14	IPE300	HEB260	12	2	yes	solid	1.2	D
CT2	IPE330	HEB260	12	3*	no	composite**	1.1+	C
SCJ3	305x165UB40	203x203UC52	10	4	no	composite	0.2	F
SCJ4	305x165UB40	203x203UC52	10	4	no	composite	1.2	H
SCJ5	305x165UB40	203x203UC52	10	4	yes	composite	1.2	D

Note-The area of mesh is included in the reinforcement percentage.

* The third bolt row is in the extended part of compression zone of end plate.
** The ribs of decking are parallel to the steel beam.
+ This value is the ratio of the area of rebars plus mesh to the area of slab if assumed solid.

(A) Excessive joint deformation
(C) Failure of slab in shear
(D) Local buckling of steel beam
(F) Fracture of slab reiforcement
(H) Buckling of column web in compression

Table 6-7, Effective length ratios for columns of the minor axis tests (uncracked section)

TEST No.	C $kNm/mrad$	SWAY MODE			NON-SWAY MODE		
		$\overline{K_r}$	η	$\frac{l}{L}$	$\overline{K_r}$	η	$\frac{l}{L}$
5	6.25	0.150	0.70	1.93	0.345	0.50	0.685
6	22.3	0.386	0.48	1.43	0.650	0.35	0.618
8	15.6	0.305	0.53	1.51	0.570	0.38	0.633

Table 6-8, Effective length ratios for columns of the minor axis tests (cracked section)

TEST No.	C $kNm/mrad$	SWAY MODE			NON-SWAY MODE		
		$\overline{K_r}$	η	$\frac{l}{L}$	$\overline{K_r}$	η	$\frac{l}{L}$
5	6.25	0.260	0.73	2.05	0.513	0.58	0.725
6	22.3	0.657	0.58	1.56	0.852	0.56	0.716
8	15.6	0.510	0.62	1.70	0.758	0.52	0.696

Table 6-9, Effective length ratios for column of the bare steel minor axis test

TEST No.	C $kNm/mrad$	SWAY MODE			NON-SWAY MODE		
		$\overline{K_r}$	η	$\frac{l}{L}$	$\overline{K_r}$	η	$\frac{l}{L}$
11	7.78	0.308	0.70	1.93	0.572	0.55	0.711
	5.07	0.225	0.76	2.19	0.465	0.61	0.742

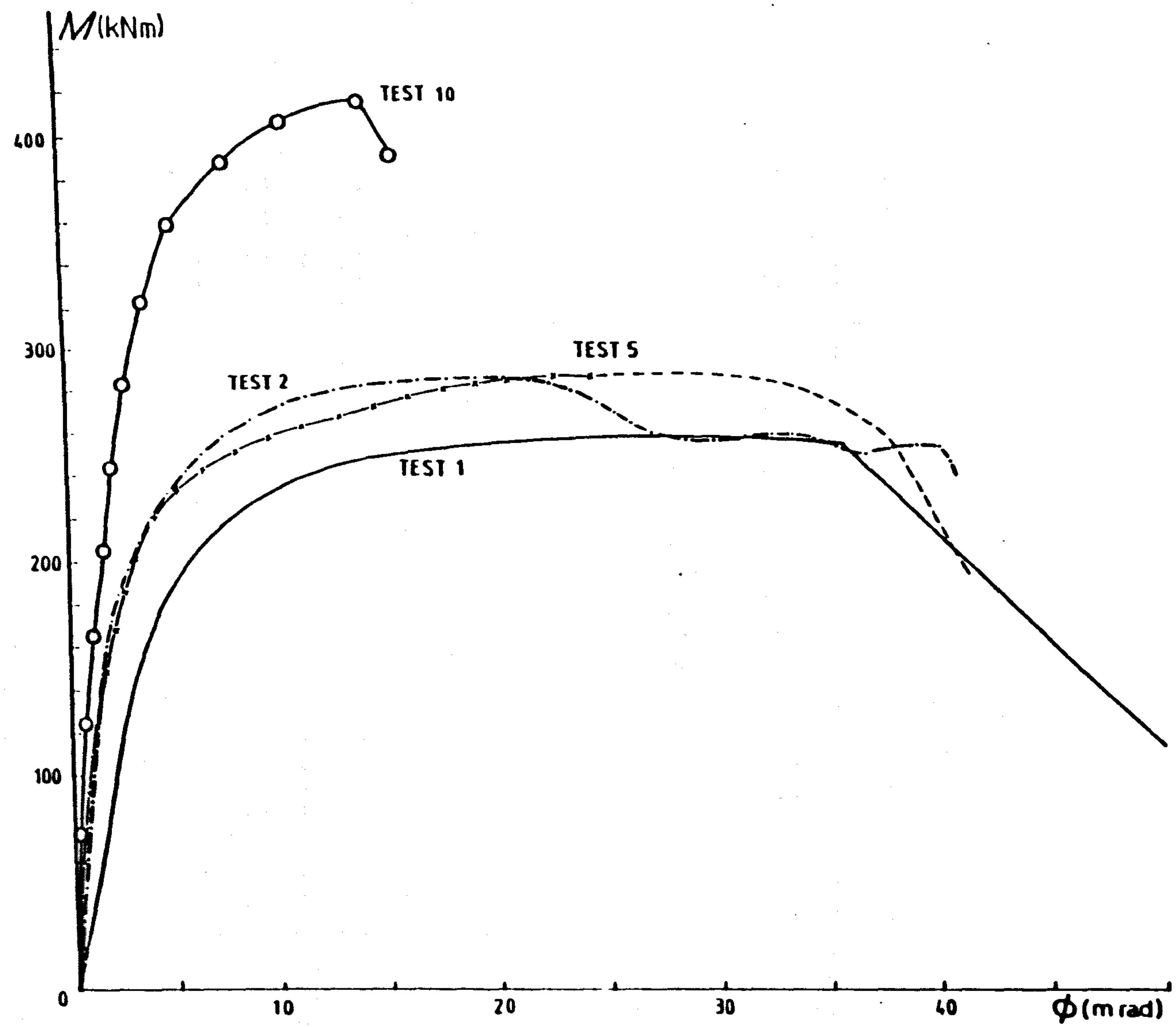


Fig. 6-1, Comparison between moment-rotation curves of connections with 1% Reinforcement

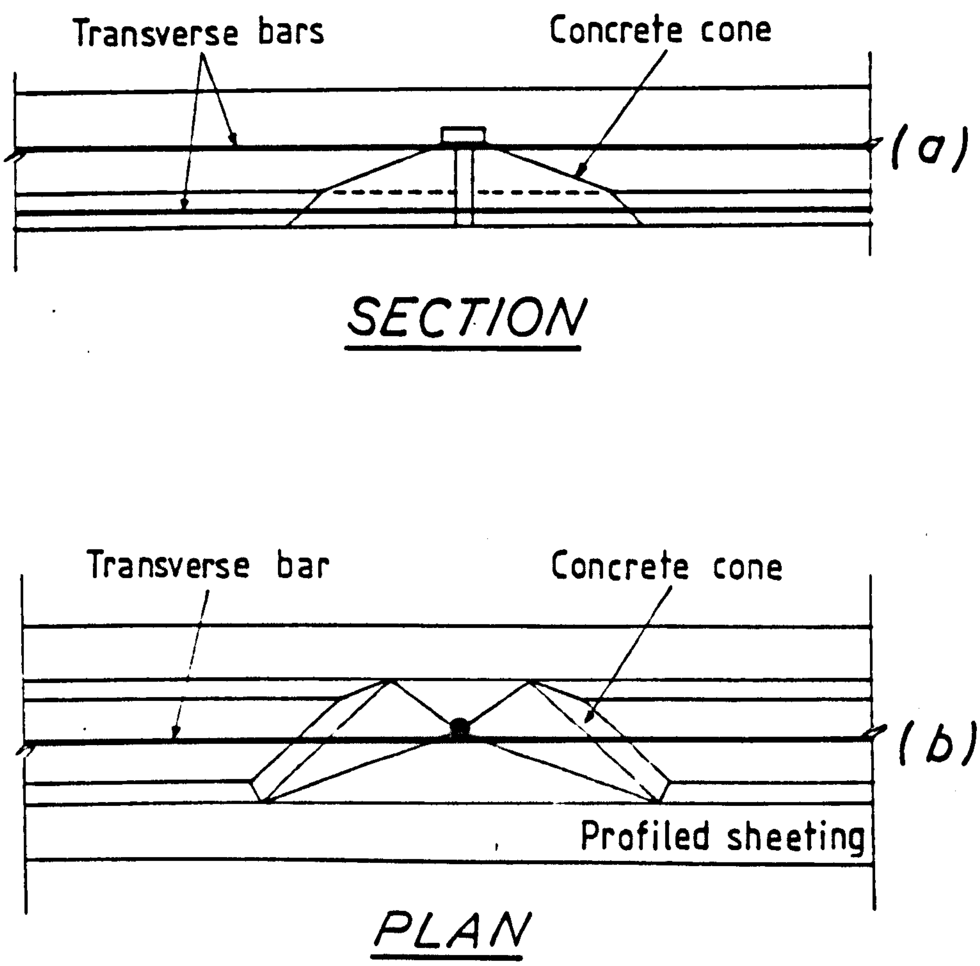
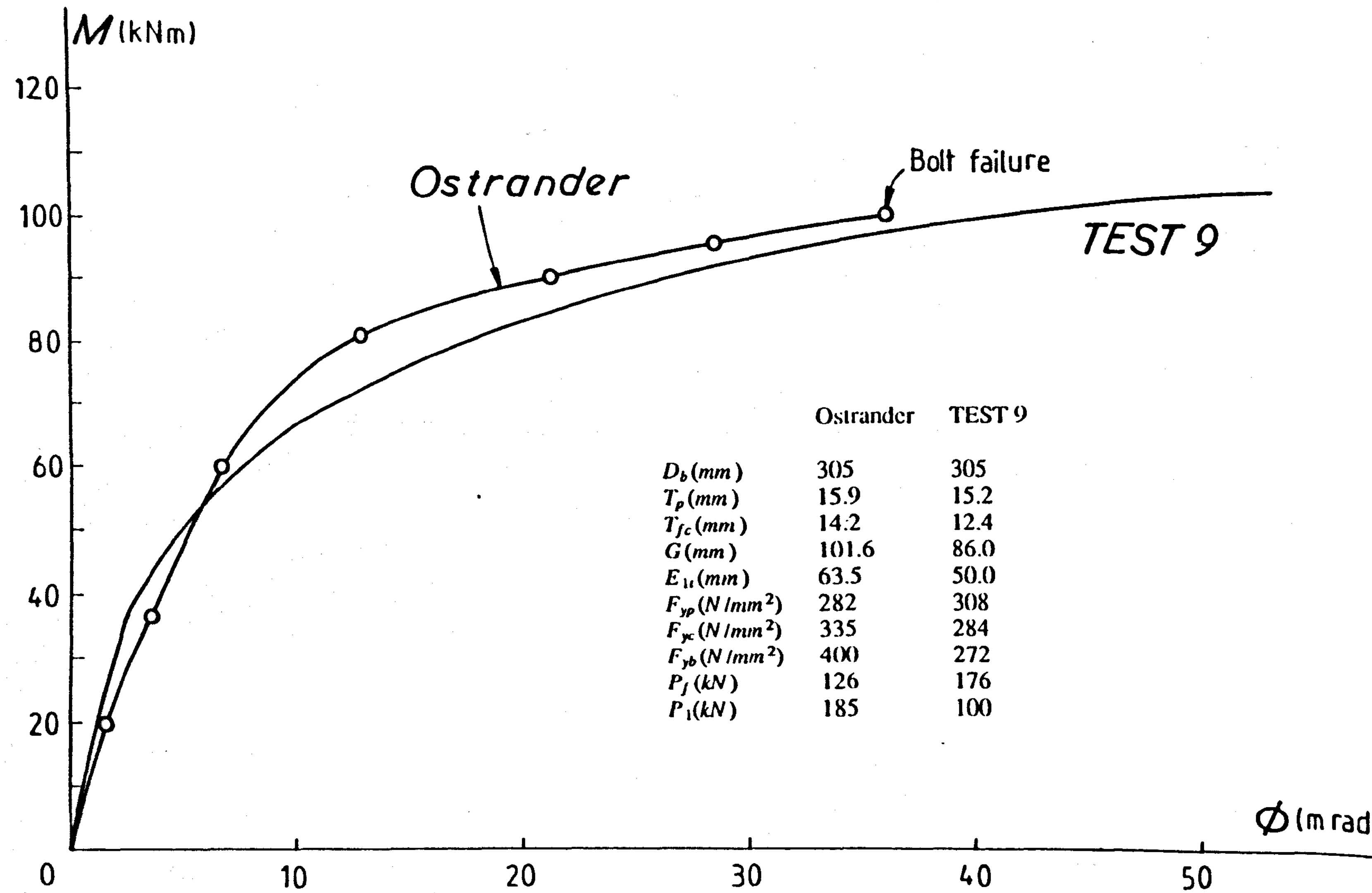


Fig. 6-2, Failure around the studs and the position of the transverse bars



	Ostrander	TEST 9
D_b (mm)	305	305
T_p (mm)	15.9	15.2
T_{fc} (mm)	14.2	12.4
G (mm)	101.6	86.0
E_{1t} (mm)	63.5	50.0
F_{yp} (N/mm^2)	282	308
F_{yc} (N/mm^2)	335	284
F_{yb} (N/mm^2)	400	272
P_f (kN)	126	176
P_1 (kN)	185	100

Fig. 6-3, Comparison between M–ϕ of Test 9 and test by Ostrander(1970)

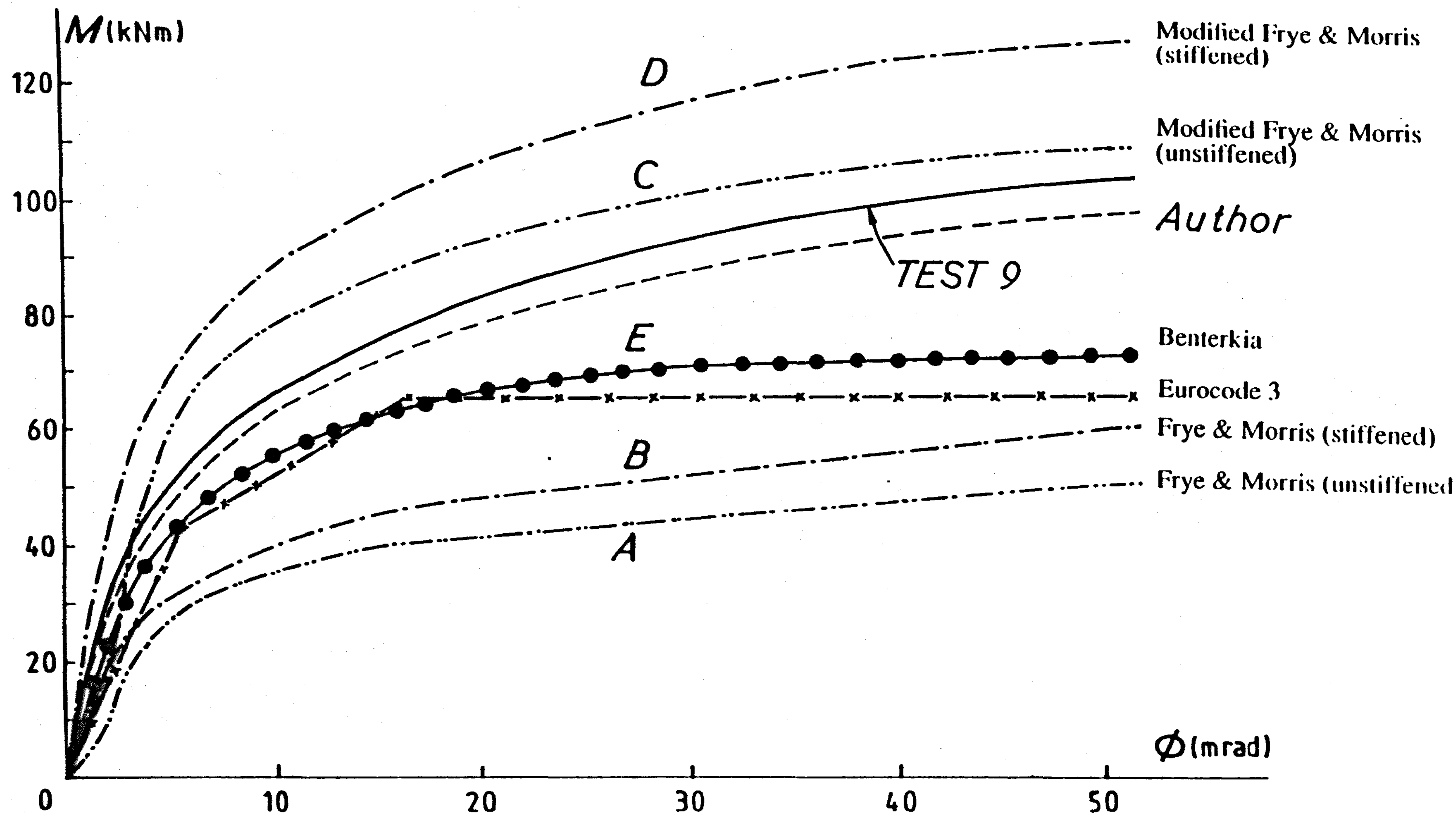


Fig. 6-4, Comparison between M–ϕ of Test 9 and the prediction methods

$$\phi = C_2 \frac{M}{C_1 - M}$$

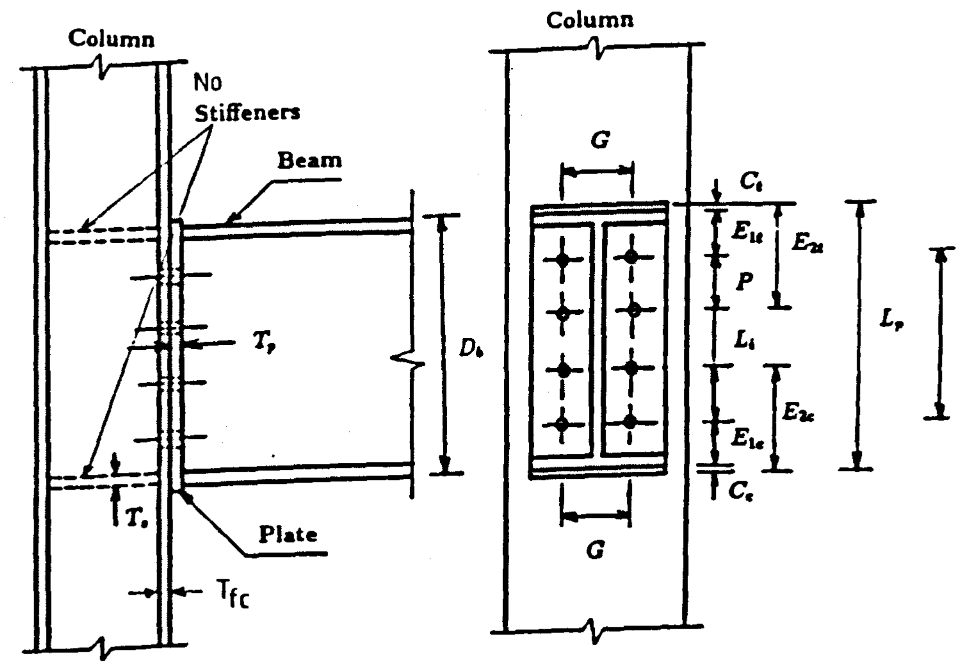
the empirical factors C_i are given by

$$C_i = D_b^{\alpha_{i,1}} \cdot T_p^{\alpha_{i,2}} \cdot T_{fc}^{\alpha_{i,3}} \cdot G^{\alpha_{i,4}} \cdot E_{1t}^{\alpha_{i,5}} \cdot E_{2t}^{\alpha_{i,6}} \cdot F_{yp}^{\alpha_{i,7}}$$

$$\cdot F_{yc}^{\alpha_{i,8}} \cdot F_{yb}^{\alpha_{i,9}} \cdot P_f^{\alpha_{i,10}} \cdot P_1^{\alpha_{i,11}}$$

where

$\alpha_{1,1} = +0.870$	$\alpha_{2,1} = -2.385$
$\alpha_{1,2} = +0.917$	$\alpha_{2,2} = +0.281$
$\alpha_{1,3} = +1.299$	$\alpha_{2,3} = +0.631$
$\alpha_{1,4} = -0.652$	$\alpha_{2,4} = -0.013$
$\alpha_{1,5} = -0.919$	$\alpha_{2,5} = -0.561$
$\alpha_{1,6} = -0.006$	$\alpha_{2,6} = -0.270$
$\alpha_{1,7} = +0.379$	$\alpha_{2,7} = -0.258$
$\alpha_{1,8} = +2.004$	$\alpha_{2,8} = -0.234$
$\alpha_{1,9} = +0.090$	$\alpha_{2,9} = +0.643$
$\alpha_{1,10} = -0.233$	$\alpha_{2,10} = +0.221$
$\alpha_{1,11} = +0.240$	$\alpha_{2,11} = +0.391$



F_{yp} = yield stress of end plate F_{yc} = yield stress of column F_{yb} = yield stress of beam

P_f = proof load of bolts P_1 = preload of the bolts

Fig. 6-5, The prediction method proposed by Benterkia(1991) for unstiffened flush end plate connection

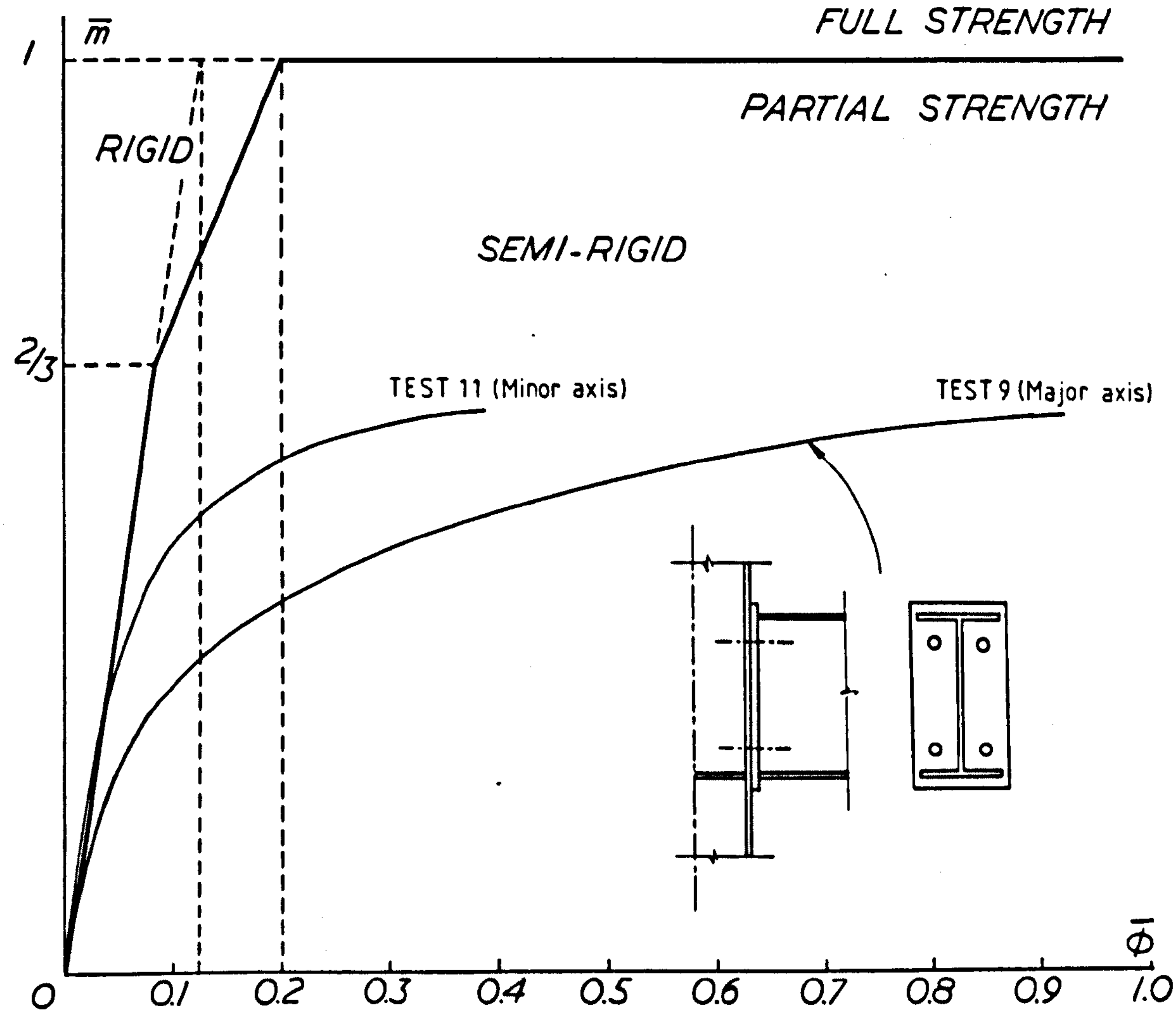


Fig. 6-6, Non-dimensional presentation of the behaviour of bare steel joints compared to the EC3 definition for rigid connections

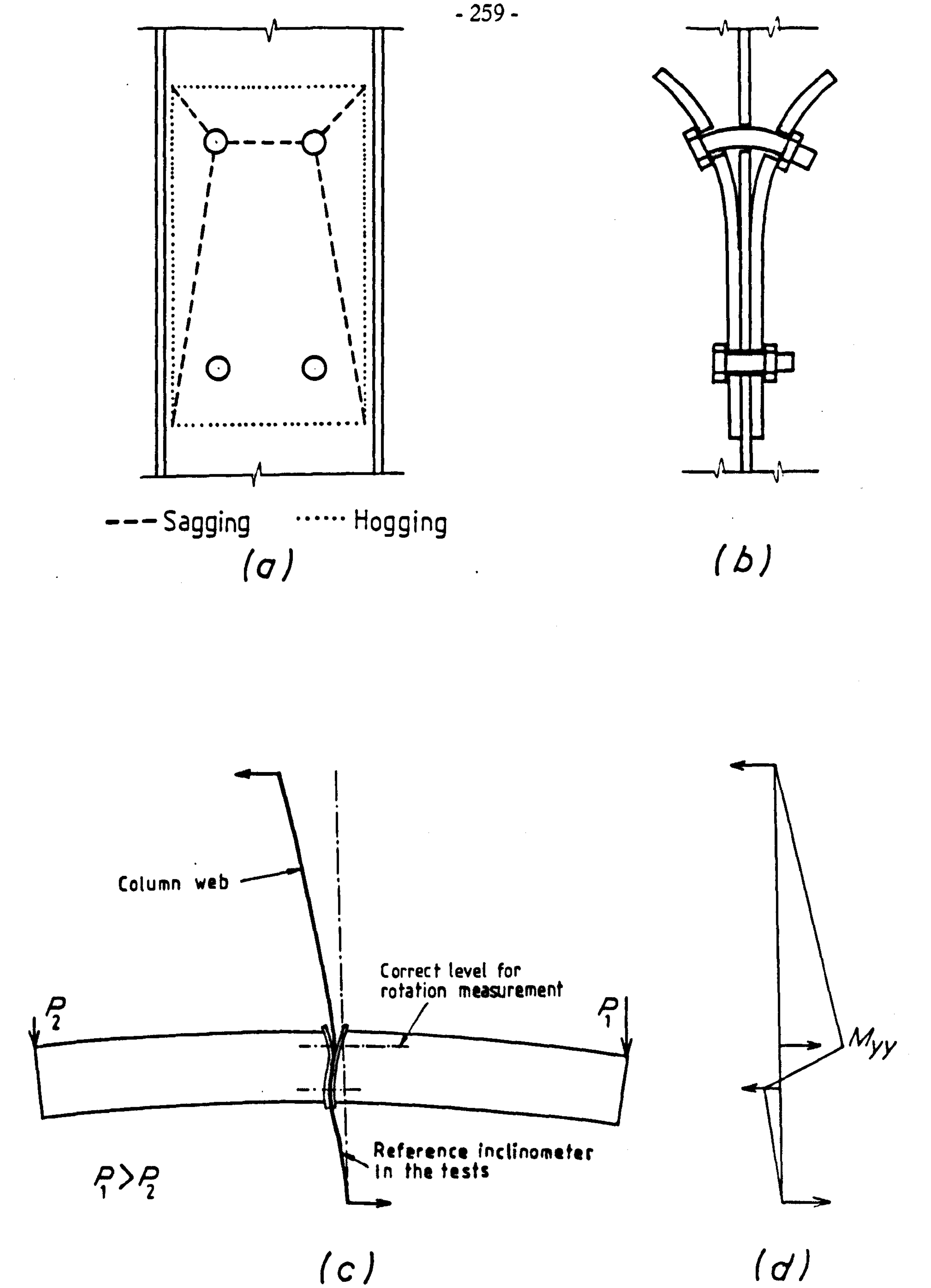


Fig. 6-7, *(a) Yield line pattern of external joint.* *(b) Deformation of end plate and bolt.*
(c) Deformed shape of column in unbalanced phase. *(d) Bending moment diagram of column.*

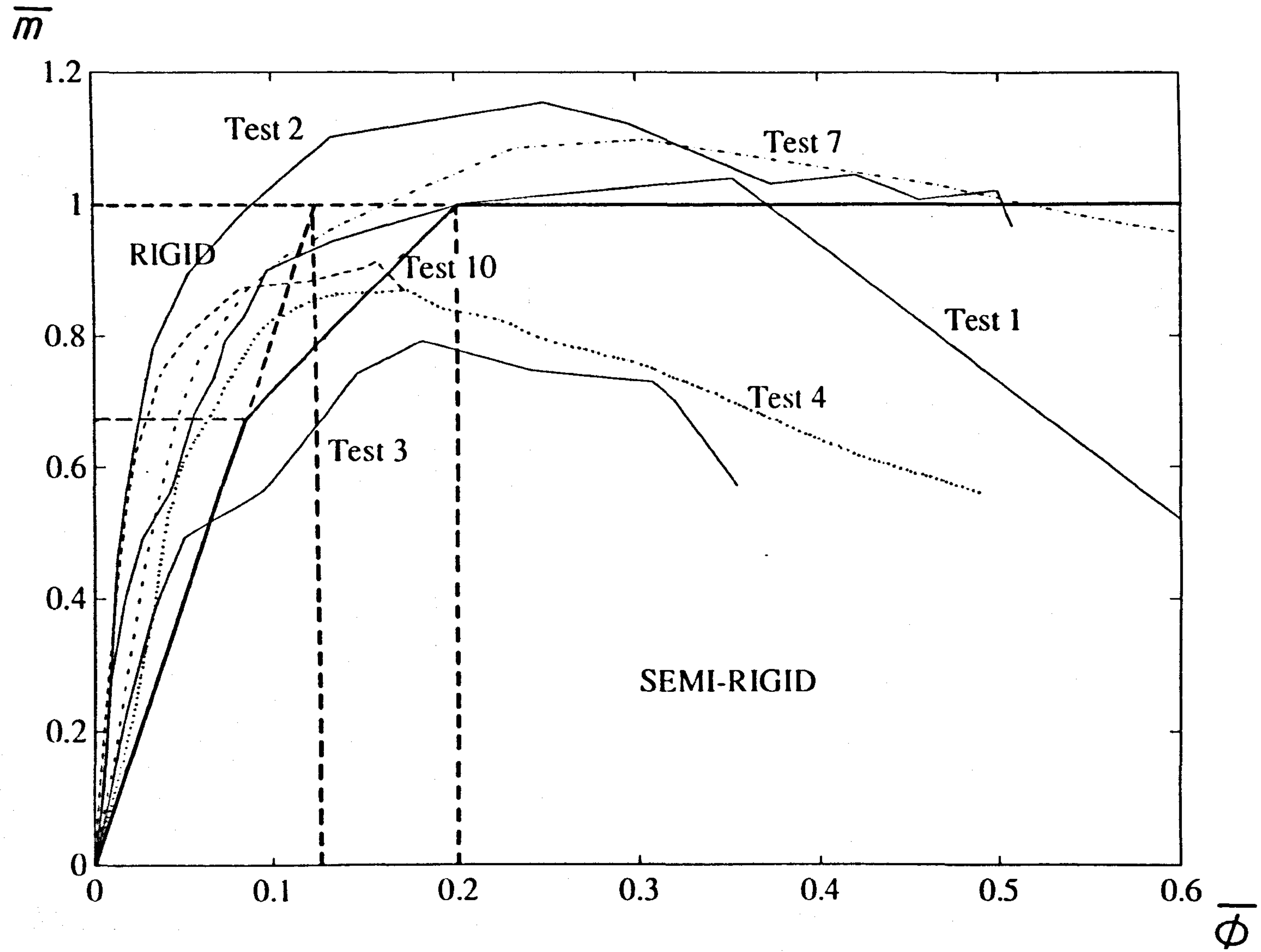
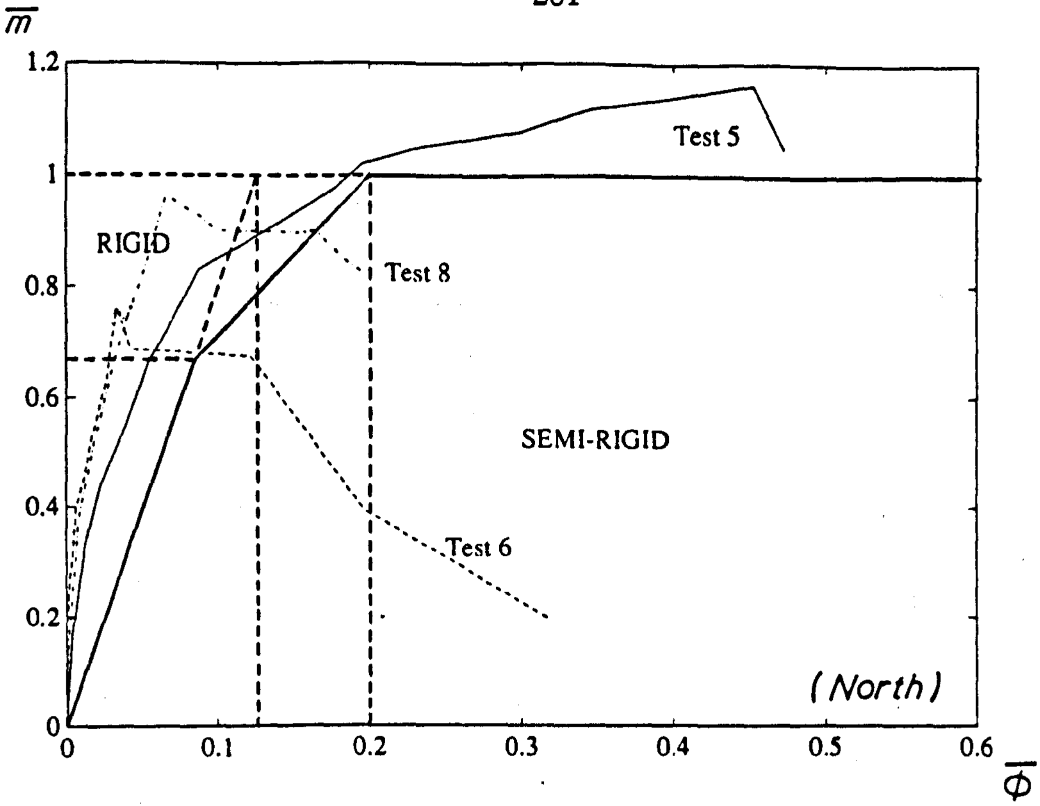
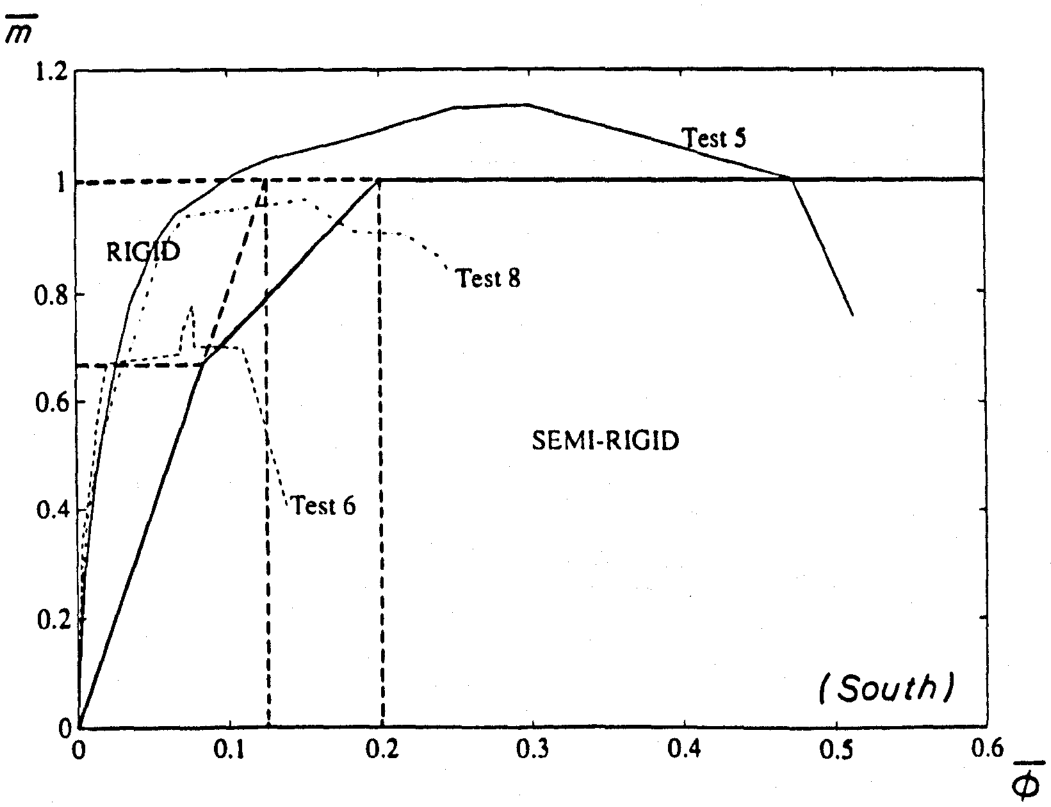


Fig. 6-8, Non-dimensional presentation of connection behaviour for major axis connections, compared to the EC4 definition of rigid joints



(a)



(b)

Fig. 6-9, Non-dimensional presentation of connection behaviour for minor axis connections, compared to the EC4 definition of rigid joints

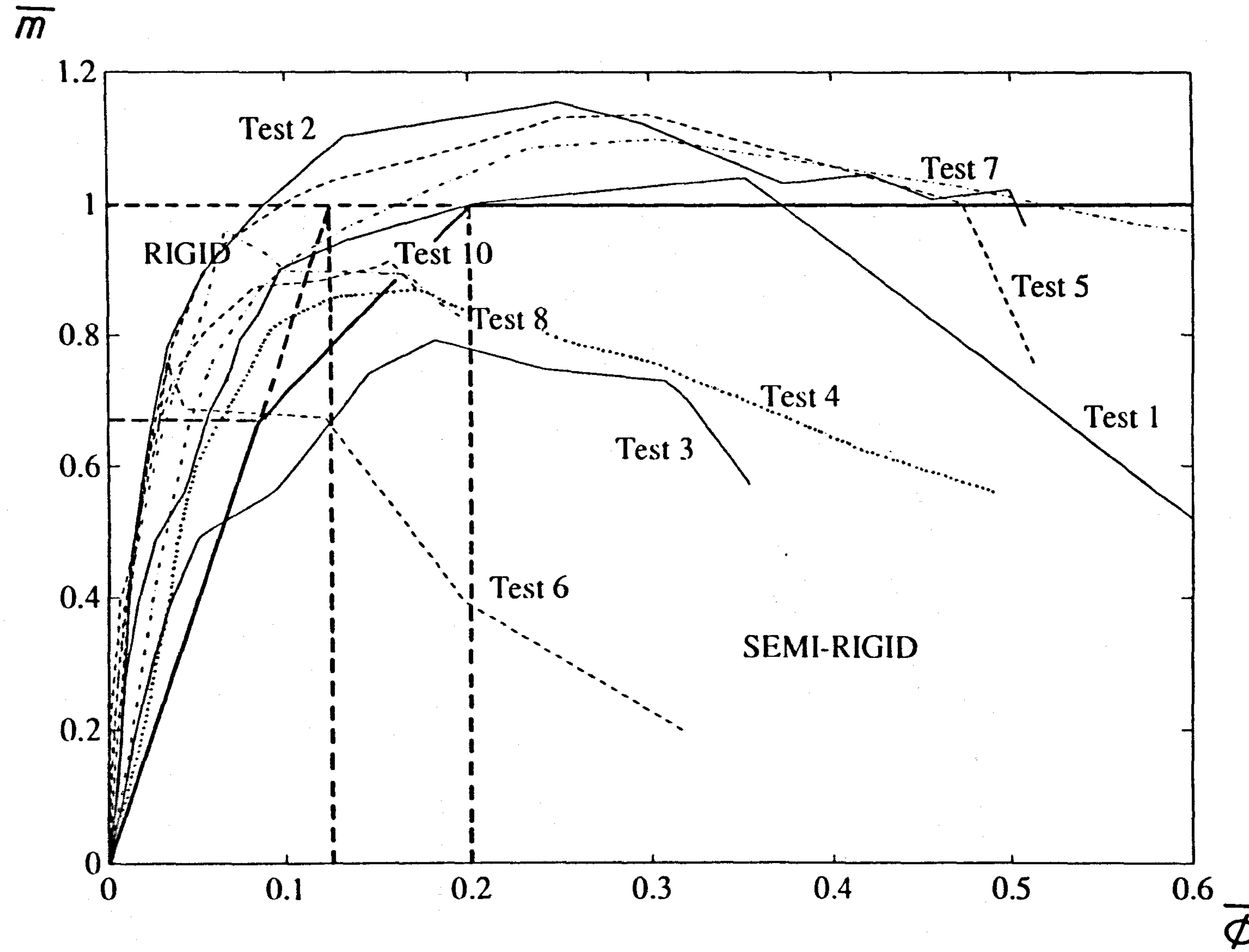


Fig. 6-10, Non-dimensional presentation of connection behaviour for all composite tests, compared to the EC4 definition of rigid joints

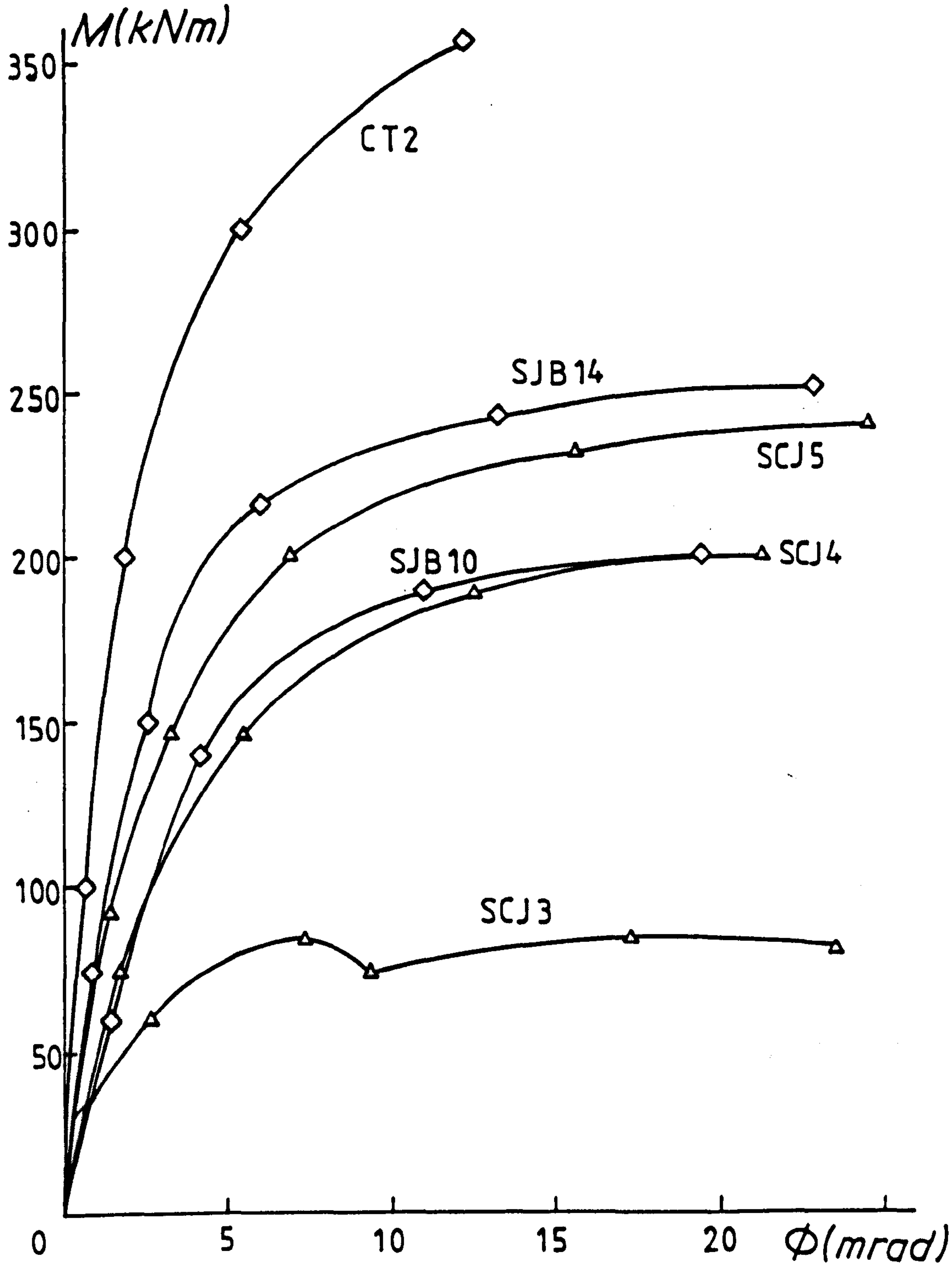


Fig. 6-11, Moment-rotation curves of tests conducted in Italy and Nottingham

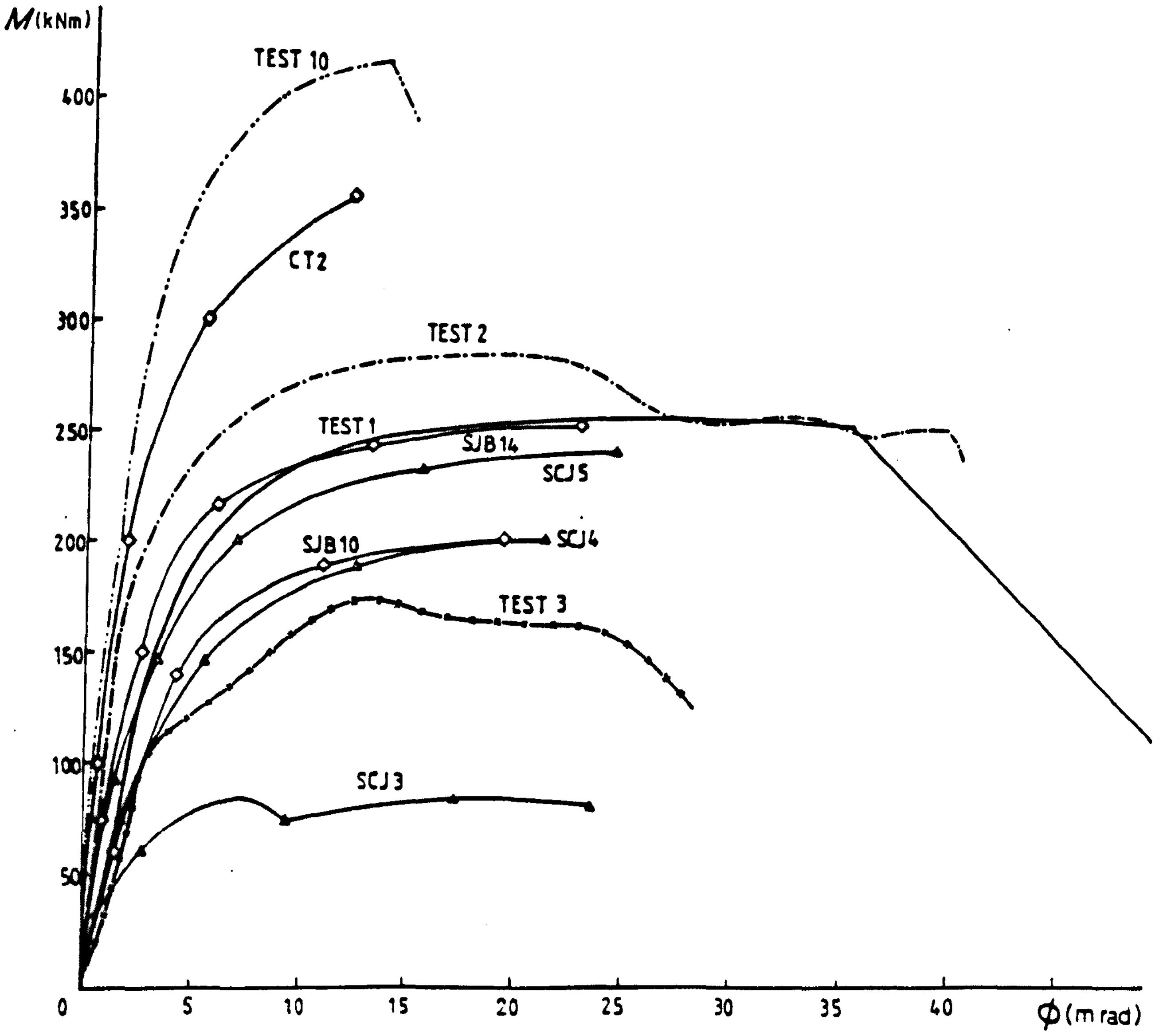


Fig. 6-12, Moment-rotation curves of tests conducted in Warwick, Italy and Nottingham

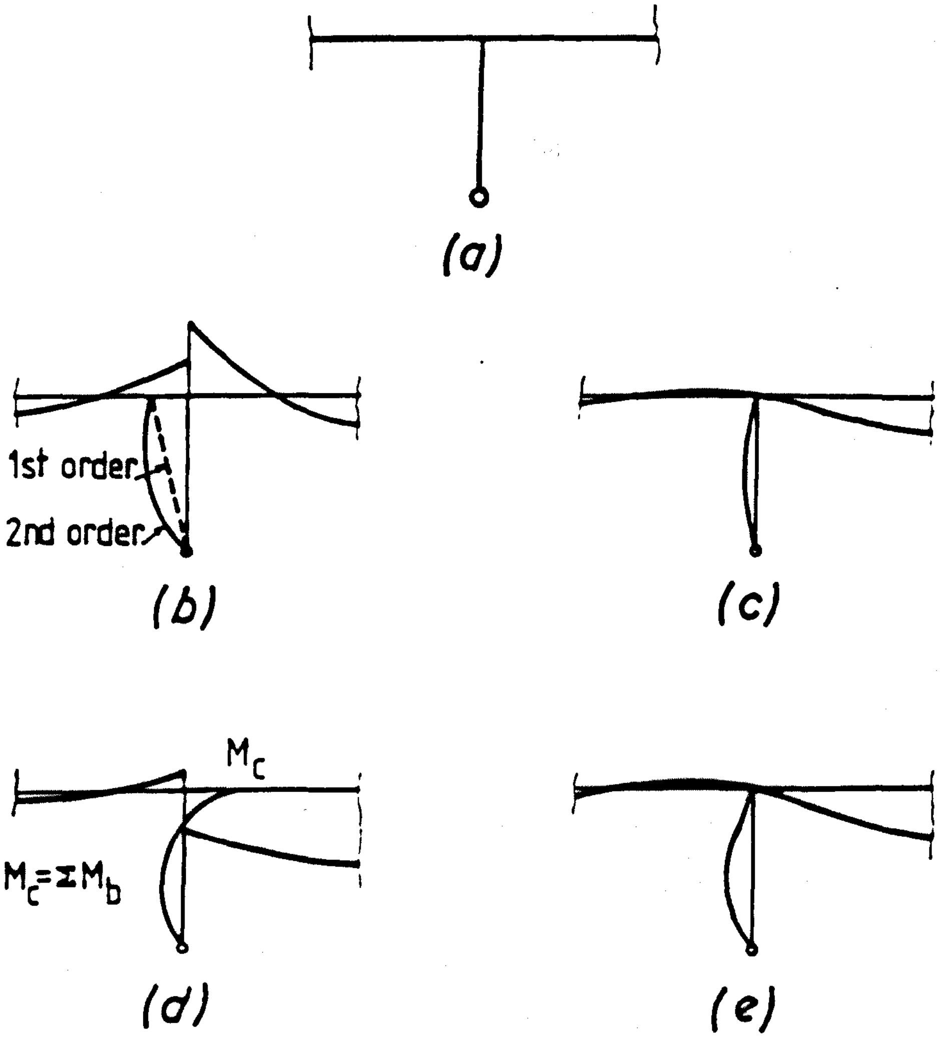
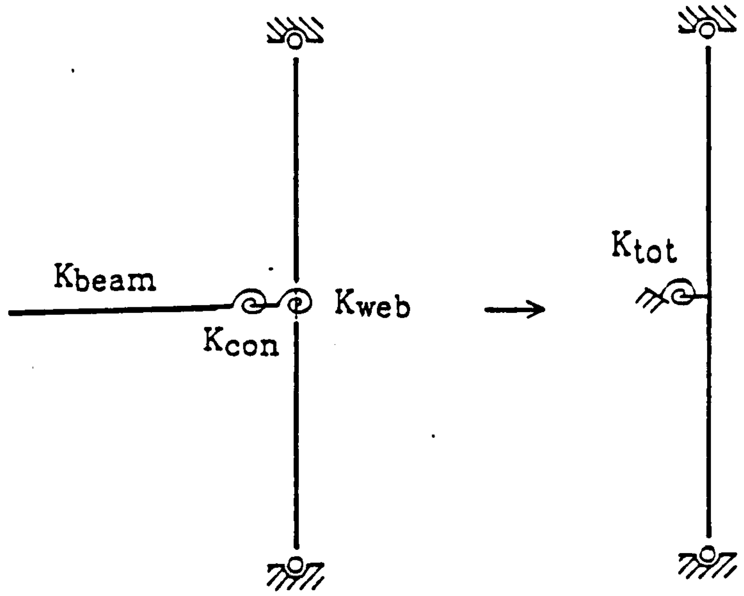


Fig. 6-13, Bending moments and deflections of a beam-to-column assembly (deflections are not proportional to moments)



$$K_{joint} = \frac{K_{con} K_{web}}{K_{con} + K_{web}}$$

where K_{con} = actual stiffness of the connection

K_{web} = stiffness of the web panel

$$K_{tot} = \frac{K_{beam} K_{con} K_{web}}{K_{beam}(K_{con} + K_{web}) + K_{con} K_{web}}$$

Fig. 6-14, Stiffness of external minor axis joint (Gibbons(1990))

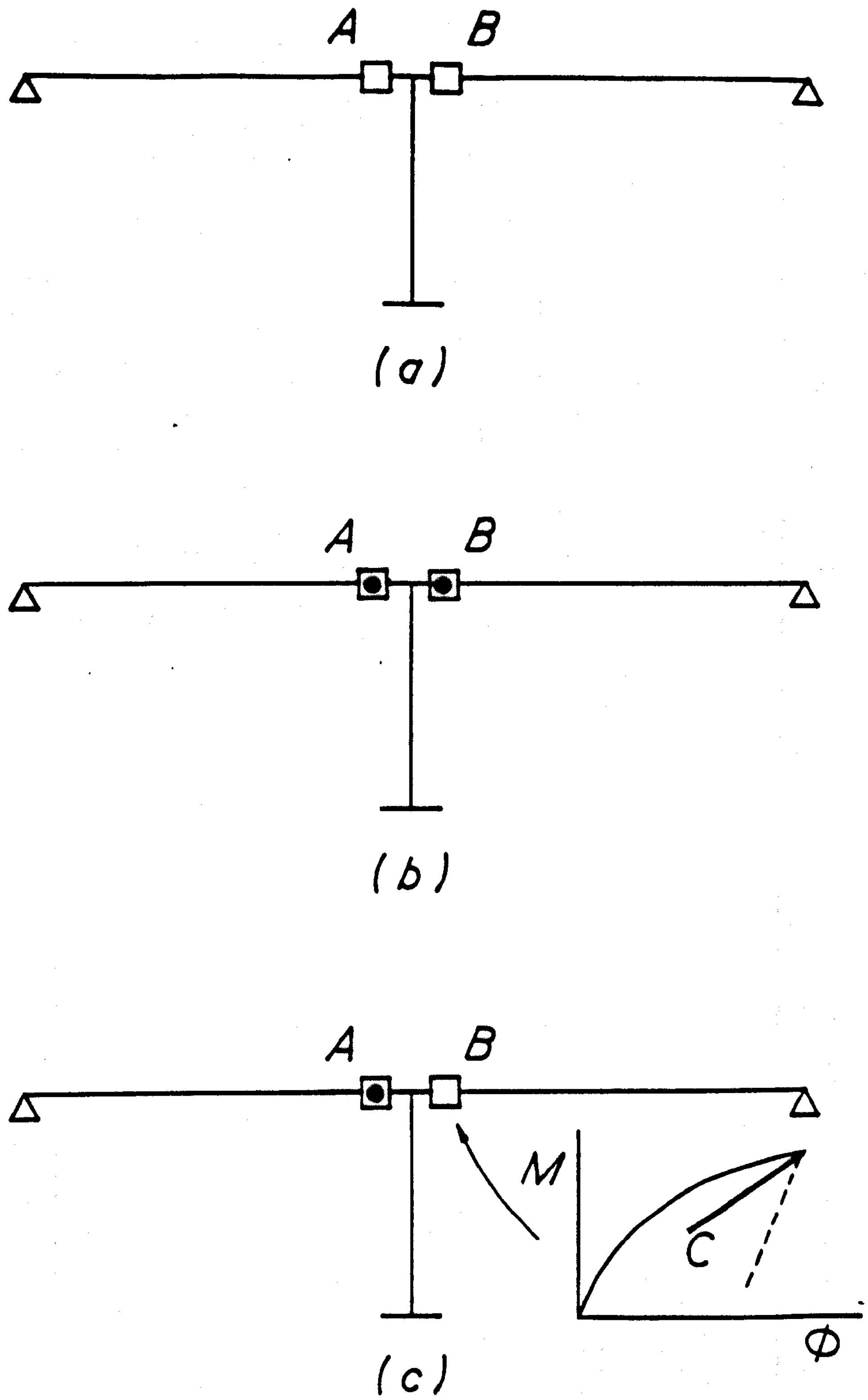
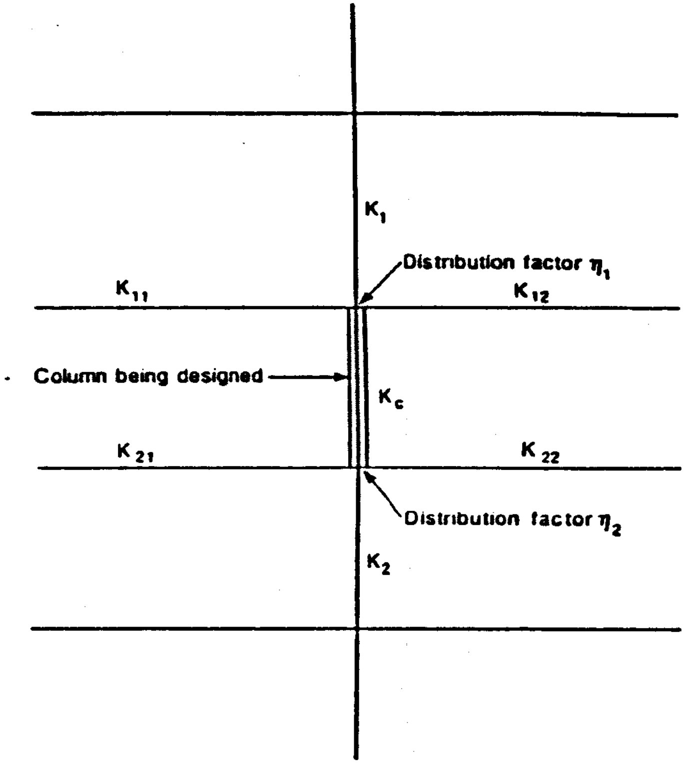


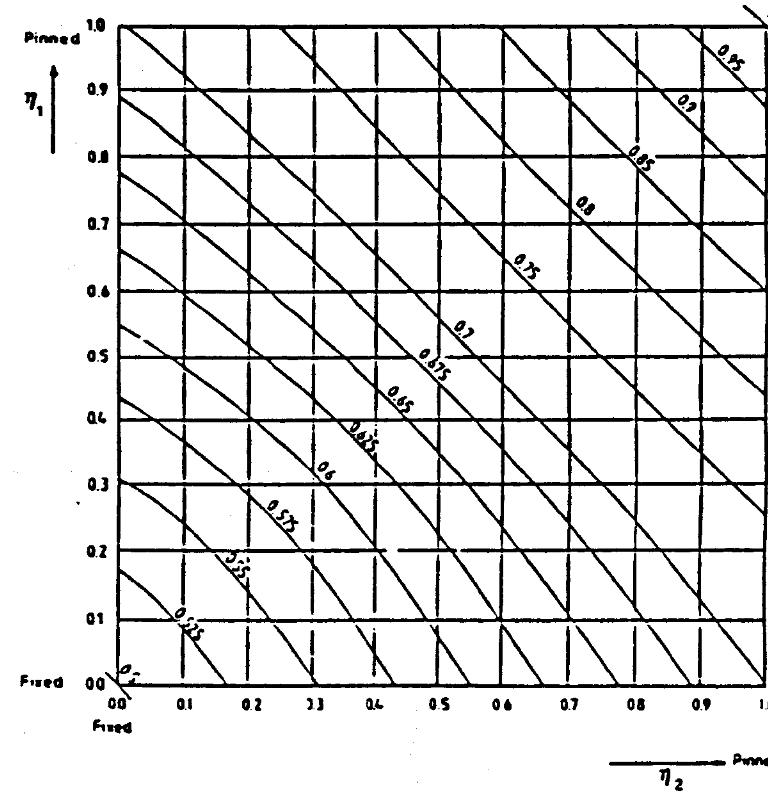
Fig. 6-15, Formation of plastic hinges and restraint provided by unloading connection



$$\eta_1 = \frac{K_c + K_1}{K_c + K_1 + K_{11} + K_{12}}$$

$$\eta_2 = \frac{K_c + K_2}{K_c + K_2 + K_{21} + K_{22}}$$

(a) Distribution factors for continuous columns



(b) Buckling length ratio l/L for a column in a non-sway mode

Fig. 6-16, Determination of effective length of column to EC3

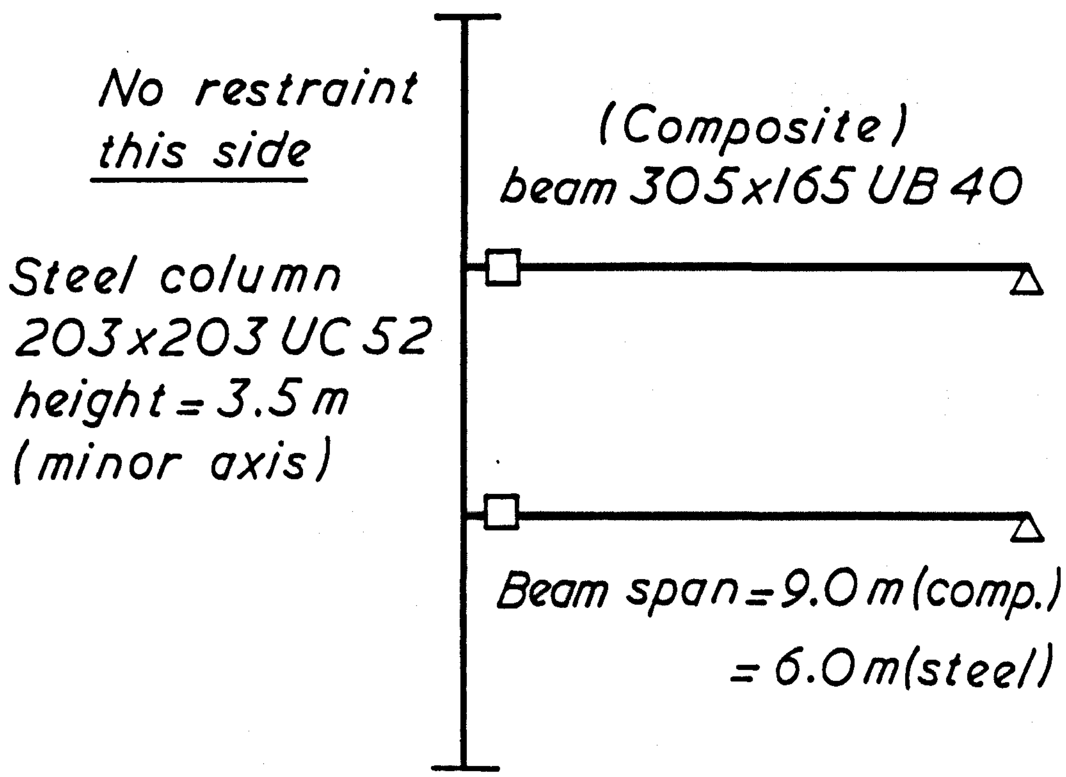


Fig. 6-17, Assemblage assumed in quantifying effective length of column

Chapter 7

PREDICTION OF CONNECTION STIFFNESS

In Chapter 2 methods for prediction of joint behaviour were discussed and various approaches used by researchers were summarized. The mechanical model was described as a promising tool for simulating the response of structural joints. These models are based on the mechanical behaviour of the material of the connection components.

In this chapter, different approaches to the prediction of behaviour of composite joints are summarized. A mechanical model is introduced in which the deformation elements of the connection are modelled as elastic-plastic springs. The mathematical expressions for the mechanical behaviour of components are given. The proposed method is examined against the tests conducted by the author. The aim is to introduce the main approach on which a practical procedure can be constructed.

7-1-Background to Prediction of Stiffness of Composite Joints

A review of the numerical analyses and prediction methods for connection stiffness has been given by Zandonini(1989). The models used by researchers can be categorized as the following:

a) Finite difference model.

b) Finite element model.

c) Mechanical model.

In these models, the connection behaviour is simplified and approximation is made for the response of key elements of the composite joint.

Echeta(1982) used a finite difference formulation. He modelled the steel connection based on the displacement boundary conditions. The shear connector flexibility was then allowed for in the composite model and the material and geometrical non-linearities were taken into account. The discrepancies between the results of the analytical model and the experimental $M-\phi$ curves of his tests have been suggested to be due to overestimation of interface slip and neglect of tensile resistance of concrete (Zandonini(1989)).

Leon & Lin(1986) used a finite element technique. The model is for a composite connection with a steel cleated joint. They determined multilinear stress-strain behaviour for materials. The contributions due to deformation of the bolts, column components and shear connectors' flexibility were neglected. Nonetheless, comparison with the experimental $M-\phi$ curves showed close agreement (Zandonini(1989)).

Tschemmernegg(1988) has extended his mechanical model for steel connections described in Chapter 2 to the particular composite joint of Fig. 7-1. In the figure, the concrete slab is not in touch with the steel column. The beam-column interaction is developed only via the steel joint. The column web is unstiffened and is assumed to be the component governing the joint behaviour. The achievements of his work on steel connections can then be applied to this composite joint, particularly the spring simulation of the column web in compression. He assumed that full shear connection is provided, and the effect of shear lag is negligible. Hence the effect of interface slip between the steel beam and the concrete slab was neglected. He has therefore effectively treated the steel joint plus the reinforcement (see Fig. 7-1(c)).

Where the column web is stiffened, attention should concentrate on other sources of flexibility such as that of the shear connectors. In the model proposed by Johnson & Law(1981), the contributions of the slab and the steel connection are taken into account.

Johnson & Law(1981) suggested a trilinear representation of the moment-rotation curve for the composite connection shown in Fig. 7-2(a). The $M-\phi$ behaviour of the joint is approximated by three phases given in Fig. 7-2(b). They determined the elastic stiffness of the joint by an elastic partial interaction analysis of the cantilever shown in Fig. 7-2(c). They assumed the centre of rotation of the connection was at the beam's bottom edge, which is supported by a rigid support (stiffened column). The rotation of the composite joint ϕ is then taken as the rotation of the beam end. They adopted the theory developed by Newmark et al(1951) in which the tensile resistance of concrete and shear lag is neglected. The general solution for the axial force F (Fig. 7-2(c)) is given by:

$$F = C_0 \sinh\sqrt{R}x + C_1 \cosh\sqrt{R}x + \frac{PQx}{R} \qquad (7.1)$$

in which:

$$R = k\overline{EI} / (s\overline{EA}\sum EI)$$

$$(7.2)$$

$$Q = kD / (s\sum EI)$$

where k is the short term stiffness of a stud connector and s is the connectors' spacing, and:

$$\overline{EA} = 1 / (\frac{1}{EA_r} + \frac{1}{EA_s})$$

$$\overline{EI} = \sum EI + \overline{EA}D^2$$

$$\sum EI = EI_r + EI_s$$

The value of C_1 is found from the end condition; at $x=0$, $F=0$ and hence $C_1=0$. The value of C_0 can be found from equilibrium and compatibility conditions at the column face where $x=L$. Assuming C_s as the known rotational stiffness of the steel connection at the level of the top bolts, from Fig. 7-2(c):

$$P\,L = M = F\,d_F + C_s\,\phi \qquad (7.3)$$

$$\phi\,d_s = \frac{\Delta\,d_s}{d_F} + \gamma \qquad (7.4)$$

where γ is the interface slip. The value of C_0 will be found from:

$$\gamma = \frac{s}{k}\,(C_0\sqrt{R}\cosh\sqrt{RL} + \frac{QP}{R}) \tag{7.5}$$

Having found the value of C_0, the joint rotation is determined as:

$$\phi = \frac{M}{C_s}\,(1-\frac{Q\,d_F}{R}) - \frac{C_0\,d_F\sinh\sqrt{RL}}{C_s} \tag{7.6}$$

The trilinear moment-rotation behaviour of the composite connection (Fig. 7-2(b)) requires the values of M_e, M_p and ϕ_p to be determined. Johnson and Law assumed that M_e is half of M_p and:

$$M_p = M_{sc} + A_r F_{yr} d_F \tag{7.7}$$

The value of ϕ_p was assumed as 10 mrad for "F-type" and 20 mrad for "P-type" joints based on the experimental results of Johnson & Hope-Gill(1972) and Law(1983). "F-type" refers to full shear interaction at the steel-concrete interface, and "P-type" refers to partial interaction. They believed that F-analysis overestimates the joint stiffness for a continuous member (compared to a cantilever) because there could be slip at the point of contraflexure. On the other hand, P-analysis underestimates stiffness. Therefore, the former was presented as an upper bound to connection stiffness and the latter as a lower bound.

In order to check this method, Johnson and Law used the results of above mentioned tests. They used the load-slip curve obtained by Menzies(1971) for the stiffness of stud connectors. Fig. 7-3 gives the comparison between theoretical and experimental results of Law's tests. The agreement of the elastic stiffness and plastic moment capacity is good, the discrepancies attributable partly to the trilinear presentation.

As described, the evaluation of ϕ_p is based on limited test results against which the method has been checked. Johnson and Law did not provide any general procedure for determining ϕ_p values. It is clear that the values of ϕ_p depend on the joint characteristics, even when the steel connection is a flush end plate, as variables of the steel joint and the slab components affect the stiffness of the connection. Therefore, a correct approach to a prediction method of connection stiffness is to account for the behaviour of the individual components of a composite joint.

7-2-Basis of Proposed Model

Recently, a model was proposed by Jaspart(1991) and Gerardy & Schleich(1991) as shown in Fig. 7-4. They argued that the shear connectors are designed to obtain a full interaction between steel and reinforced concrete members. This is the case usually assumed for the hogging region of composite continuous beams. They then concluded that the resulting absence of slip allows one to assume the in-plane indeformability of the end sections of the composite beam. The model shown in Fig. 7-4(b) is based on this assumption. It consists of an infinitely rigid beam. The end section of the beam lies on an elastic-plastic foundation represented by axial springs simulating the deformation and the resistance of:

1) the reinforcing bars;
2) the concrete;
3) the tension components of the steel joint;
4) the compression components of the steel joint.

The deformation of components in 3 and 4 above consists of contribution from the bolts in shear, tension and bearing; the column flange; the column web; the cleat and slip between the cleat and the steel beam.

Each of the springs shown in Fig. 7-4(b) is characterized by a specified non-linear force-displacement curve. The methods of modelling have already been proposed for each source of deformability by ARBED Report(1991). The moment-rotation curve corresponding to a particular composite connection is built up step by step by distributing, at each level of bending moment, the loads between the springs according to their actual relative stiffness.

Gerardy and Schleich have applied this procedure to composite connections with cleated joints and different amounts of reinforcement. The presence or absence of slip between the cleats and the beam have been examined. The results are shown in Fig. 7-5. A close agreement has been reported between the experimental and theoretical curves, except for the prediction of the ultimate resistance in Fig. 7-5(b) in which the mode of failure was the buckling of the column web, compared to the yielding of rebars in Fig. 7-5(a).

Since in such models, the individual behaviour of the connection components can be expressed by mathematical expressions, which are based on the mechanical properties of the relevant material, the method represents a valuable tool for predicting the connection response. This method can be employed in the computer analysis of structural connections integrated with any analysis program for frames.

7-3-Proposed Model

The present author has extended the method proposed by Jaspart(1991) and Gerardy & Schleich(1991) to composite connections with steel end plate joints as shown initially in Fig. 7-6.

Each spring in Fig. 7-6 simulates the stiffness of a component of the connection and can be assumed to be elastic or elasto-plastic. The rotational stiffness of the steel connection at any level of applied moment can be converted to the stiffness of compression spring, K_a, and tension spring, K_b. Where the column web is stiffened, K_a equals infinity.

The terms K_c, K_r, K_s and K_p in Fig. 7-6 correspond respectively to the contributions of concrete, reinforcement, shear connectors and profiled steel sheeting to the stiffness of composite connection. Full interaction is assumed to exist at the interface between the steel beam and the concrete slab.

7-3-1-Determination of Spring Stiffness of Steelwork Joint

The rotational stiffness of the steelwork connection is:

$$C_s = \frac{M_s}{\phi_s} = \frac{F_b\, D_b}{\phi_s} \tag{7.8}$$

The joint rotation ϕ_s can be written as:

$$\phi_s = \frac{\Delta_b}{D_b} \tag{7.9}$$

where Δ_b is the extension of spring b, i.e. the displacement at the level of the top bolt row in Fig. 7-6. From Eqns. (7.8) and (7.9), the stiffness of spring b can be written as:

$$K_b = \frac{F_b}{\Delta_b} = \frac{C_s}{D_b^{\,2}} \tag{7.10}$$

The stiffness C_s can be calculated for steelwork end plate joints from the methods explained in Chapter 2.

7-3-2-Initial Model

For the first instance, it is assumed that plane sections remain plane and contributions of concrete in tension and the steel sheeting are negligible. The assumption of plane sections requires that the slip at the interface of steel beam and slab is very small and therefore can be neglected. Accordingly, the model is simplified to springs K_a, K_b and K_r as shown in Fig. 7-7. The deformation of joint at the level of top row of bolts is:

$$\Delta_b = \frac{F_b}{K_b} \tag{7.11}$$

Using the Hooke's law:

$$\Delta_r = \frac{F_r}{K_r} = \frac{F_r\, l}{E_r\, A_r} \tag{7.12}$$

where l is the assumed length of reinforcing bars having extension Δ_r. Hence:

$$K_r = \frac{E_r\, A_r}{l} \tag{7.13}$$

The values of F_b and F_r can be found from equilibrium and compatibility conditions in terms of the applied moment:

$$F_b = \frac{D_b\, K_b}{D_b^2 K_b + D_r^2 K_r} \,.M \tag{7.14}$$

$$F_r = \frac{D_r\, K_r}{D_b^2 K_b + D_r^2 K_r} \,.M \tag{7.15}$$

The results of the tests on composite connections can be used to examine this model. Tests 1 and 3 (with 1% and 0.5% reinforcement respectively) are considered for which the stiffness of their steel connection, C_s, is available from Test 9. Assuming M is equal to half of the maximum test moment, the value of K_b is found from Eqn. (7.10). The value of K_r is calculated from Eqn. (7.13) assuming:

$$l = \frac{D_c}{2}$$

where D_c is the depth of the column section. The value of F_r is found from Eqn. (7.15) and used in Eqn. (7.12) to determine the value of Δ_r. The connection rotation is finally calculated from:

$$\phi = \frac{\Delta_r}{D_r} \tag{7.16}$$

The calculated rotations are compared with the experimental rotations in Table 7-1. The values of F_r found for Tests 1 and 3 (given in the table) are greater than those found from using the actual recorded strains at the same level of moment. It is seen from Table 7-1 that the calculated values of rotation are much smaller than the experimental values. Therefore, the connection flexibility has been underestimated. It is concluded that the flexibility of shear connectors cannot be excluded from the derivation of connection stiffness. Hence, the problem lies within finding a suitable load-slip curve for the stud connectors used in the tests.

7-3-3-Determination of Spring Stiffness of Shear Connectors

The available test data of the load-slip behaviour of welded studs was reviewed including those given in Table 7-2. Three push out tests by Mottram & Johnson(1990) have some similarity to the shear connection in the author's tests. These are shown in Fig. 7-8. The most suitable curve is H30-1-F refering to Super Holorib profiled sheeting, Grade 30 concrete, one stud per trough in a "favourable" position. The approximated bilinear elastic-plastic behaviour plotted in this figure is then used as the measure of connector's flexibility with an ultimate slip equal to 10 mm (Mottram & Johnson(1990)). The elastic stiffness of a stud is:

$$K_s = \frac{P_k}{\gamma_p} \tag{7.17}$$

The values of P_k = 100 kN and γ_p = 0.5 mm were taken, which result in K_s = 200 kN/mm.

The sum of the shear force in the studs of a cantilever is equal to the longitudinal force in the reinforcement. The slip at the connection depends on the nearest stud to the column. Under increasing load, it is assumed that the first stud provides resistance to slip, until it becomes plastic. Its force then remains constant and equal to its maximum resistance. Additional load is then assumed to be resisted by the next stud deforming elastically until the plastic resistance of that stud is attained. Further load will then be carried by the next stud and so forth. This theory is based on the observation of progressive formation of transverse and longitudinal cracks along the cantilever beam specimens, starting from the column region progressing towards the free end of the cantilever.

7-3-4-Connection Model Including Slip

To model the real behaviour of the composite connection, two of the slab components are now considered. These belong to the reinforcing bars and the shear connectors. It should be noted that the contribution of concrete in tension has not yet been taken into account.

The reinforcement in the slab limits the joint's deformation. The force in the reinforcement is transferred to the studs via the surrounding concrete. The flexibility of the studs contributes to the deformation of composite connection which is associated with horizontal slip at the interface of the steel beam and the concrete slab. Therefore, a model in which these two actions are accounted for will be that shown in Fig. 7-9.

The equilibrium and compatibility equations (assuming $\Delta_r = \frac{F_r}{K_r}$, $\Delta_b = \frac{F_b}{K_b}$ and $\Delta_s = \frac{F_s}{K_s}$ where $F_s = F_r$) can be written as:

$$F_r\, D_r + F_b\, D_b = M$$

$$\frac{\Delta_b}{D_b} = \frac{\Delta_r}{D_r} + \frac{\Delta_s}{D}$$

which gives:

$$F_b = \frac{\left(\frac{1}{K_r D_r} + \frac{1}{K_s D}\right) D_b\, K_b}{D_r + \left(\frac{1}{K_r D_r} + \frac{1}{K_s D}\right) D_b^2\, K_b} \cdot M \tag{7.18}$$

$$F_r = \frac{1}{D_r + \left(\frac{1}{K_r D_r} + \frac{1}{K_s D}\right) D_b^2\, K_b} \cdot M \tag{7.19}$$

From geometry, the rotation of composite connection is expressed as:

$$\phi = \frac{\Delta_r}{D_r} + \frac{\Delta_s}{D} \tag{7.20}$$

The procedure for checking the above approach against the results of Tests 1 and 3 is similar to that explained in 7-3-1. The calculated values for a moment equal to half of the maximum test moments are given in Table 7-3 in comparison with the experimental rotations. A good agreement is seen between test and theory. It should be noted that the contribution of mesh reinforcement has not been accounted for in the calculations. The greater rotations resulted from calculations could be considered to be due to neglect of stiffness provided by mesh.

Table 7-4 therefore gives a comparison between calculated rotations, excluding and including mesh, and experimental values. Tests 5 and 8 (minor axis with 1% and 0.5% reinforcement respectively) are also used to check the proposed model for which the stiffness of steel connection has been taken from Test 11. Since two different values of stiffness for the north and south connections are obtained in the bare steel tests; the smaller values are used and the calculated rotations are compared to the test results of composite connections on the side with greater rotations. As seen in Table 7-4, the increase in the amount of reinforcement by including mesh has a limited influence on the stiffness of connection particularly for the minor axis connections. Therefore, another source of stiffness, which is the contribution of concrete in tension, should be taken into account.

7-3-5-Connection Model Including Concrete

Fig. 7-10 depicts the $M-\phi$ and $M-\varepsilon$ behaviour of a composite section under negative moment (Johnson & Allison(1981)). It is assumed that rotation of the section and strain at the top of the concrete slab are both zero just after the concrete hardens. Lines OA and OB show the uncracked and cracked $M-\phi$ relations for the section. The dashed line shows the behaviour that would be expected in a test. This can be simplified by assuming that the effects of tension stiffening disappear by the time a moment M_r has been reached. Therefore, the theoretical response follows the lines OADB.

This theory can be extended to composite connections subjected to negative bending. The behaviour of the connection is initially uncracked, changing to cracked when the effects of tension stiffening disappear.

Assuming an uncracked section, the stiffness provided by the concrete slab in tension can be simulated as an additional source of stiffness associated with the reinforcement. The total stiffness of reinforcement and concrete in tension can be written by applying the Hooke's Law:

$$K_{r,c} = \frac{E_r A_r + E_c A_c}{l} \tag{7.21}$$

Taking the modular ratio $\alpha_e = \frac{E_r}{E_c}$, the above equation will be written as:

$$K_{r,c} = \frac{E_r}{l} (A_r + \frac{A_c}{\alpha_e}) \tag{7.22}$$

The value of $K_{r,c}$ can be calculated for the connections with 1% and 0.5% reinforcement (only, without mesh) taking A_c as the area of concrete above decking. The results of including the contribution of uncracked concrete are tabulated in Table 7-5 compared to the results of assuming cracked section.

The rotations of the connections at greater moments than half of the maximum test moments are also calculated. Two levels of moment are considered:

1) Two-thirds of the maximum test moment, using Table 6-2.
2) The calculated design moment resistance of the connection, taken from Table 6-1 (rebars only).

The calculated rotations assuming uncracked concrete with α_e=10 are tabulated in Table 7-6 in contrast with the experimental rotations. It is concluded from the table that general agreement exists between the theoretical and experimental values for both levels of moment. The smaller values of rotation resulted from calculations compared to the test values would be due to the assumption of uncracked concrete which would not be true at such levels of moment.

The concrete can be assumed fully cracked at the design moment resistance of the connection. The value of K_r from Eqn. (7.13) and F_r from Eqn. (7.19) can then be used. The calculated rotations for cracked concrete at a moment equal to the design resistance moment of the connections are compared to those of uncracked concrete and to the experimental rotations in Table 7-7. It is seen that the different assumptions concerning the concrete have a very limited influence on the calculation of rotations.

The elastic uncracked stiffness is based on assuming that the connection components, i.e. steelwork components and reinforcement, have not yielded. It is possible to check this assumption with the test observations and the experimental moment-strain curves of reinforcement.

The deformation of column flange in Test 1 was first observed at 200 kNm, slightly below the design moment resistance. In Test 3, this deformation was first visible at 172 kNm, much higher than the design moment resistance. The deformation of end plate in Tests 5 and 8 could only be seen at moments higher than the design moment resistance (242 kNm in Test 5 and 186 kNm in Test 8 compared to 225 kNm and 157 kNm respectively). It is concluded therefore that yielding of the steelwork components has not yet been developed at the design moment resistance of composite connection, and elastic behaviour can be assumed for these components.

The M-ε curves of Tests 1 and 3 are given in Figs. E-15 and E-17 of Appendix E respectively. The design moment resistance of the composite connection lies at the non-linear (elasto-plastic) part of these curves. It implies that some yielding had occurred in rebars before the design moment resistance was reached. Therefore, the assumption of elastic behaviour for the connection components ignores the yielding of reinforcement.

Where a computer program is written for prediction of connection stiffness, the real behaviour of materials is input. The force in an individual component is calculated from equilibrium and compatibility conditions, and the associated deformation is found from the material behaviour. Nevertheless, the simplifying assumption of elastic uncracked stiffness does not result in a significant discrepancy between the results of the author's method and experiments, although a better prediction would be expected if the elasto-plastic behaviour of materials is taken into account. Hence, for the sake of convenience in design, the composite connection may be assumed uncracked with elastic linear behaviour for materials at moments smaller than the design moment resistance of the composite connection.

7-4-Proposed Moment-Rotation Curve

The moment-rotation behaviour of composite connections could be approximated to a trilinear presentation as shown in Fig. 7-11 if cracked concrete resulted in closer values of rotation to the experimental values. However, the most significant factor in the calculation of rotation is the stiffness of shear connectors. Therefore, it is likely that the trilinear presentation is preferable where more accurate data of the behaviour of shear connectors is available. This kind of approximation would then have three phases:

1) The elastic phase, represented as the uncracked stiffness up to the half of the design moment resistance of the composite connection.

2) The transition phase, from uncracked at the end of phase 1, to cracked at the design moment resistance of composite connection.

3) The plastic phase, with zero stiffness at a constant moment equal to the design moment resistance, associated with increasing rotation up to the rotation capacity of connection.

Because of the very limited difference in the calculated rotations assuming cracked and uncracked concrete, the author suggests that a bilinear presentation is chosen. The two phases of such approximation will then be:

1) The elastic cracked phase up to the design resistance moment of the composite joint associated with the rotation:

$$\phi = \frac{\left(\frac{1}{K_r D_r} + \frac{1}{K_s D}\right)}{D_r + \left(\frac{1}{K_r D_r} + \frac{1}{K_s D}\right) D_b^2 K_b} . M \tag{7.23}$$

2) The perfectly plastic phase with zero stiffness at a constant moment equal to the design moment resistance, associated with increasing rotation up to the rotation capacity of connection.

The moment-rotation curves of Tests 1, 3, 5 and 8 are plotted according to the proposed bilinear presentation in Figs. 7-12 to 7-15 in comparison with the experimental curves. Satisfactory agreement is achieved between the theoretical approximation and the experimental curves.

7-5-Summary and Conclusions

The moment-rotation behaviour of composite connections is proposed to be approximated by a bilinear presentation with a plateau at the design moment resistance of the composite joint starting at a rotation found from Eqn. (7.23). The values of individual stiffness are found as follows:

The design resistance moment is calculated according to the formulae given in Chapter 6.

The value of K_b which is the spring stiffness representing the steelwork connection is calculated from Eqn. (7.10) using an appropriate stiffness C_s for the steel joint. This stiffness may be found using the method given in Annex J of EC3 or any suitable prediction method. Relevant approaches have already been mentioned in Chapter 2. The author used the EC3 method which has been reported to underestimate joint stiffness (Chapters 3 and 6). The EC3 method can also be used for extended end plate joints (Test 2). The resulting curves for Tests 1-3 are given in Figs. 7-16 to 7-18.

The values of K_r and $K_{r,c}$ are calculated respectively from Eqns. (7.13) and (7.22). The area of concrete is taken as that above decking within the effective breadth of the slab.

The value of K_s is determined from a suitable load-slip curve obtained either from push-out tests or an approriate prediction formula.

By applying the proposed approach to the tests conducted by the author on major and minor axis specimens with 1% and 0.5% reinforcement, the following can be concluded:

The proposed model produces satisfactory results. The predicted stiffness of the composite connections are in all cases less than the stiffness of the joints observed in the tests for moments less than about the two-thirds of the design resistance moments. Hence, the proposed method gives slightly conservative results for semi-continuous design at serviceability. The calculated maximum moment resistances were found to be normally lesser than the test values as reported in Chapter 6. Therefore, the method is also safe for plastic design at the ultimate limit state.

Table 7-1, Calculated rotations at a moment equal to half of the maximum test moment (initial model) compared to experimental rotations

TEST No.	K_b kN/mm	K_r kN/mm	M kNm	F_r kN	Δ_r mm	ϕ mrad	ϕ_{test} mrad
1	240	1770	131	320	0.18	0.46	3.00
3	240	890	90	208	0.23	0.60	2.40

Table 7-2, Push out tests reviewed for determination of load-slip behaviour

AUTHOR	TOTAL No. OF TESTS	LIGHT WEIGHT OR NORMAL WEIGHT CONCRETE	SOLID OR COMPOSITE SLAB	STUD DIAMETER (*mm*)	f_{cu} (N/mm^2)	HEIGHT OF DECKING (*mm*)
Menzies(1971)	39	LW,NW	S	19	15.2-61.0	-
Ollgaard et al(1971)	48	LW,NW	S	16,19	21-41	-
Oehlers(1980)	53	NW	S	13,19,22	25-69	-
Roik & Hanswille(1983)	25	NW	S	19,22	33-56	-
Hawkins & Mitchell(1984)	13	NW	S,C	19	24.0-37.3	38,76
Jayas & Hosain(1988)	18	LW,NW	S,C	16	24.4-34.5	38,76
Lungershausen(1988)+	46	NW	C	19,22	20.6-41.9	51-136
Robinson(1988)	49	NW	C	19	20*	51,76
Mottram & Johnson(1989)	35	LW,NW	C	19	24.1-41.5	46,51,60
Harding(1990)	24	LW,NW	S,C	19	13.4-46.6	46-60
Lloyd & Wright(1990)	42	NW	C	19	35*	50
Rakib(1991)	3	LW	C	19	45.3**	50

Note-Total No. of tests covers all tests including those comprising shear connectors other than studs, also more than one stud in a trough.
+ Decking with holes to allow welding of studs directly to the steel beam.
* Design strength.
** Average value.

Table 7-3, Calculated rotations at a moment equal to half of the maximum test moment (model including slip) compared to experimental rotations

TEST No.	K_b kN/mm	K_r kN/mm	M kNm	F_r kN	Δ_r mm	Δ_s mm	ϕ $mrad$	ϕ_{test} $mrad$
1	240	1770	131	200	0.11	1.00	3.57	3.00
3	240	890	90	133	0.15	0.67	2.60	2.40

Table 7-4, Calculated rotations at a moment equal to half of the maximum test moment (model including slip) for both rebars and rebars plus mesh compared to experimental rotations

TEST No.	Reinforcement percentage	M kNm	ϕ_R $mrad$	ϕ_{R+M} $mrad$	ϕ_{test} $mrad$
1	1.0%	131	3.57	3.52	3.00
3	0.5%	90	2.60	2.52	2.40
5	1.0%	147	3.41	3.36	3.50
8	0.5%	104	2.47	2.43	1.50

Table 7-5, Calculated rotations at a moment equal to half of the maximum test moment (model including slip and concrete) for uncracked concrete compared to cracked concrete and experimental rotations

TEST No.	M kNm	$\phi_{uncracked}$ mrad	$\phi_{cracked}$ mrad	ϕ_{test} mrad
1	131	3.41	3.57	3.00
3	90	2.36	2.60	2.40
5	147	3.25	3.41	3.50
8	104	2.30	2.47	1.50

Table 7-6, Calculated rotations at higher moments for uncracked concrete (model including slip and concrete) compared to experimental rotation

TEST No.	M = 2/3 of maximum test moment			M = design moment resistance		
	M kNm	$\phi_{calculated}$ mrad	ϕ_{test} mrad	M kNm	$\phi_{calculated}$ mrad	ϕ_{test} mrad
1	175	4.59	4.60	219	5.71	6.90
3	119	3.12	3.60	150	3.88	6.80
5	195	4.36	5.90	225	4.93	8.60
8	138	3.05	2.80	157	3.45	3.40

Table 7-7, Calculated rotations at the design moment resistance for cracked concrete compared to uncracked concrete and experimental rotations

TEST No.	M *kNm*	$\phi_{cracked}$ *mrad*	$\phi_{uncracked}$ *mrad*	ϕ_{test} *mrad*
1	219	5.98	5.71	6.90
3	150	4.28	3.88	6.80
5	225	5.18	4.93	8.60
8	157	3.76	3.45	3.40

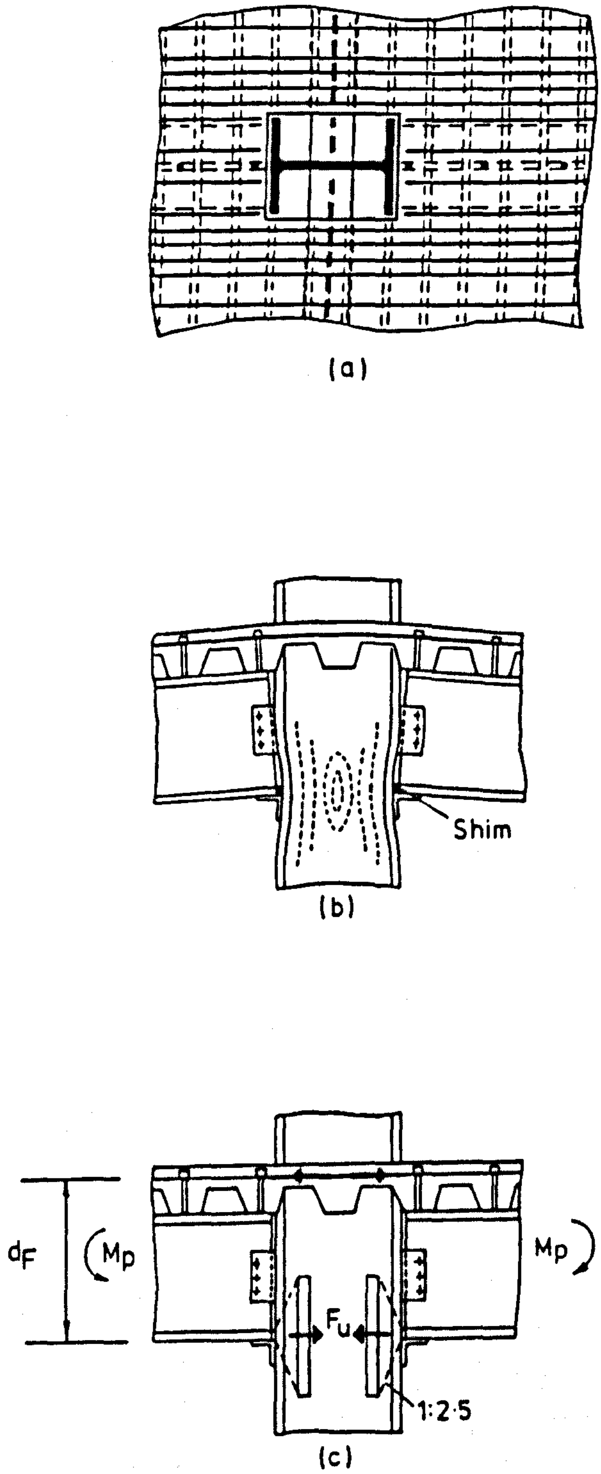
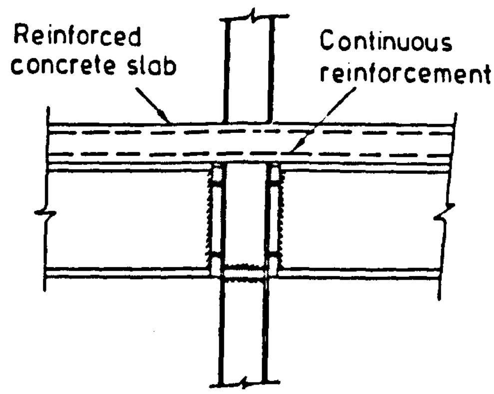
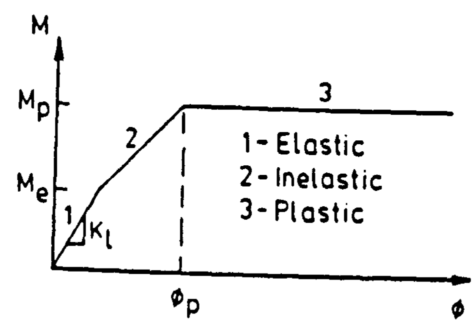


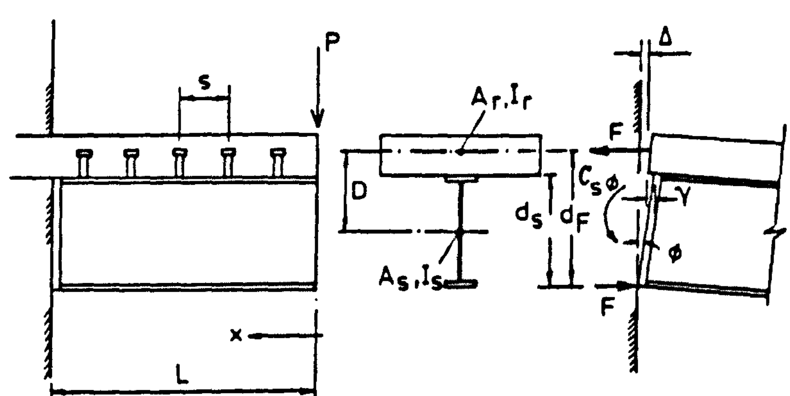
Fig. 7-1, Mechanical model proposed by Tschemmernegg(1988)



(a)

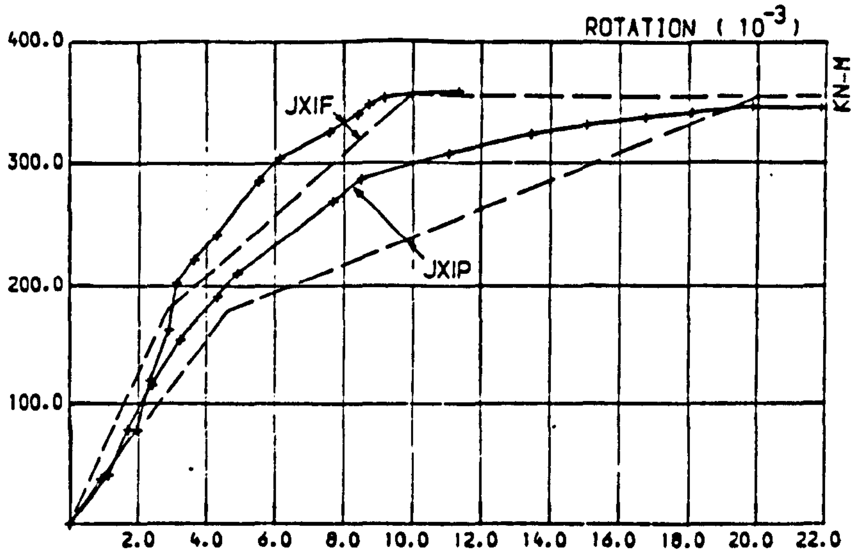


(b)



(c)

Fig. 7-2, Model proposed by Johnson & Law(1981)



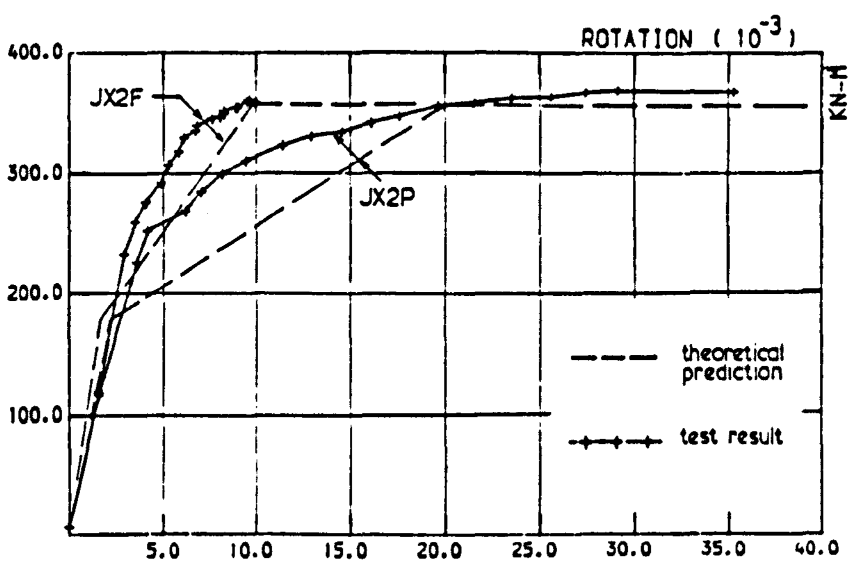
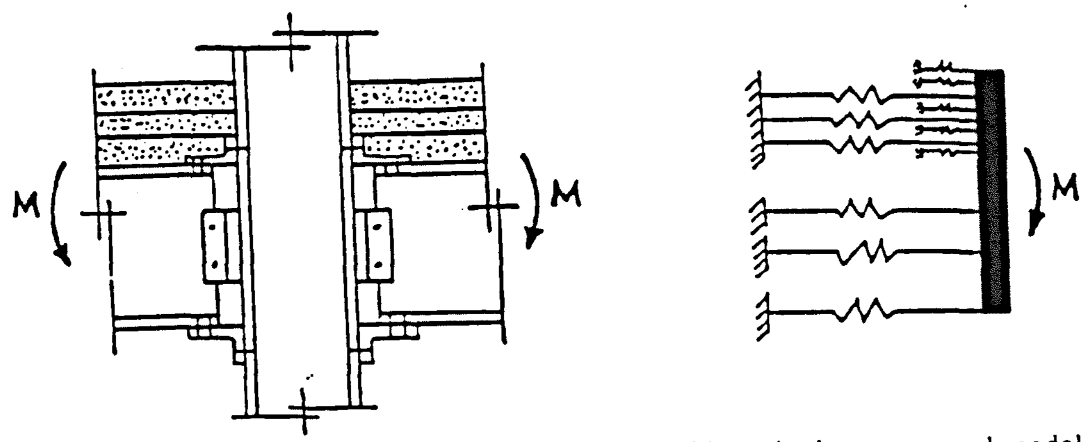
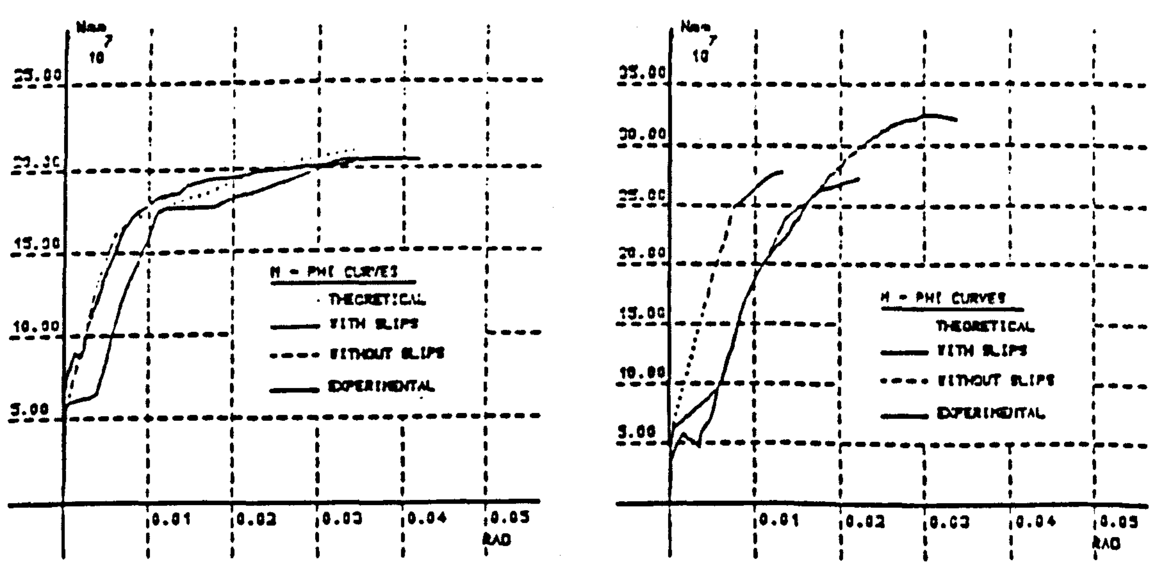


Fig. 7-3, Comparison between theoretical and experimental results (Johnson & Law(1981))



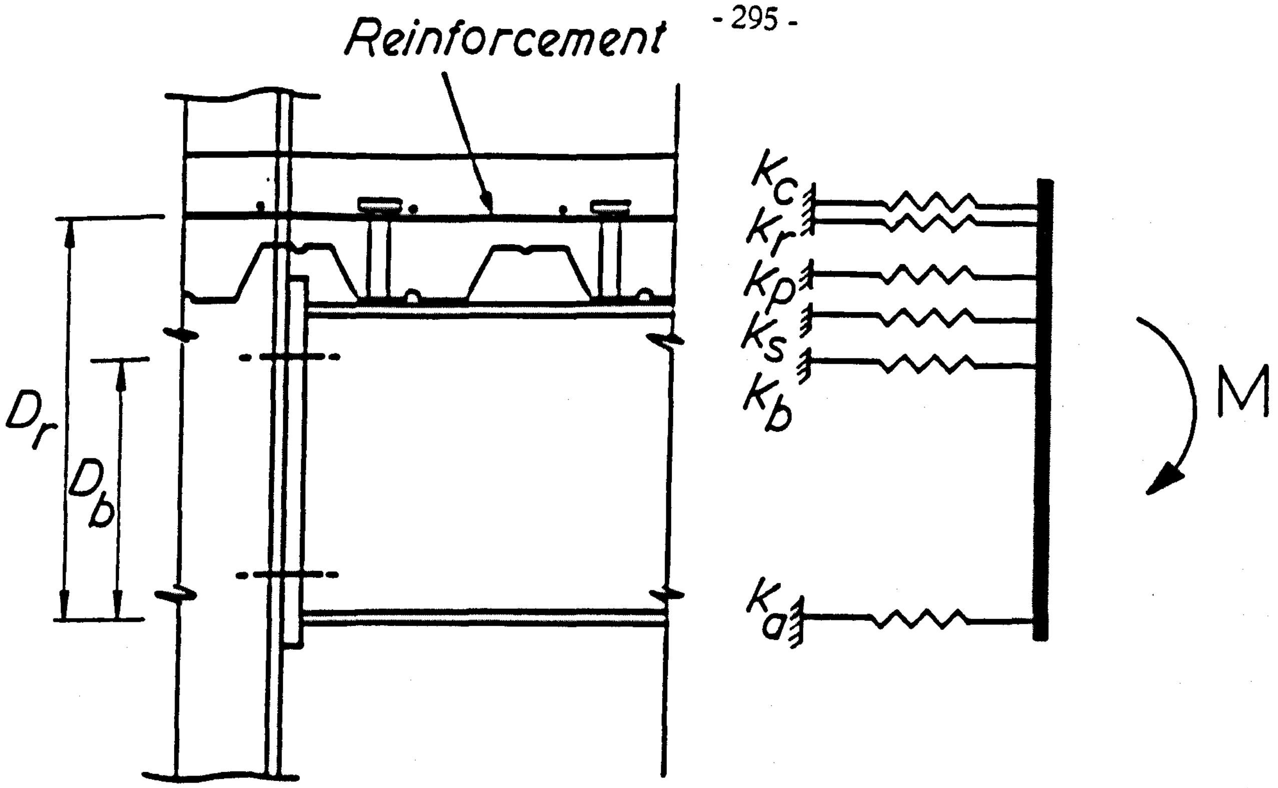
a) actual composite connection b) equivalent structural model

Fig. 7-4, Mechanical model proposed by Jaspart(1991) and Gerardy & Schleich(1991)



a) rebars of 10 mm b) rebars of 18 mm

Fig. 7-5, Comparison between theoretical and experimental results (Gerardy & Schleich(1991))



Note-For simplicity in derivation of relationships for connection stiffness, D_r is taken as the distance between the centre of bottom beam flange and centroid of reinforcement.

Fig. 7-6, Proposed model for end plate connections

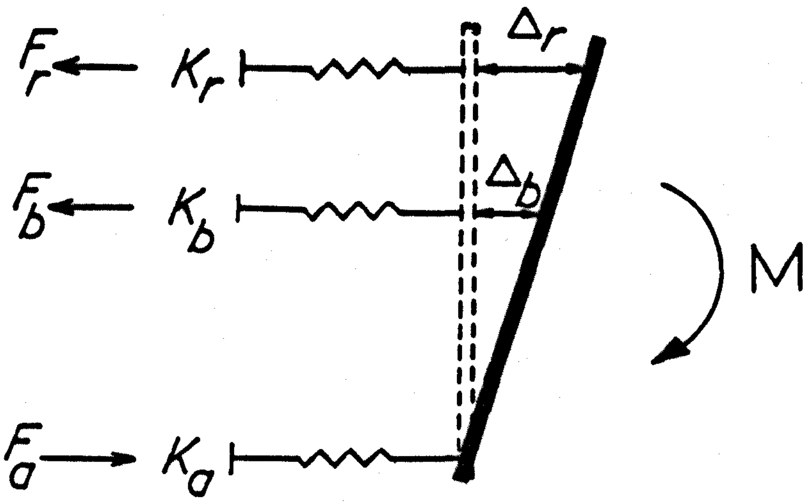


Fig. 7-7, Simplified model including steelwork connection and reinforcement

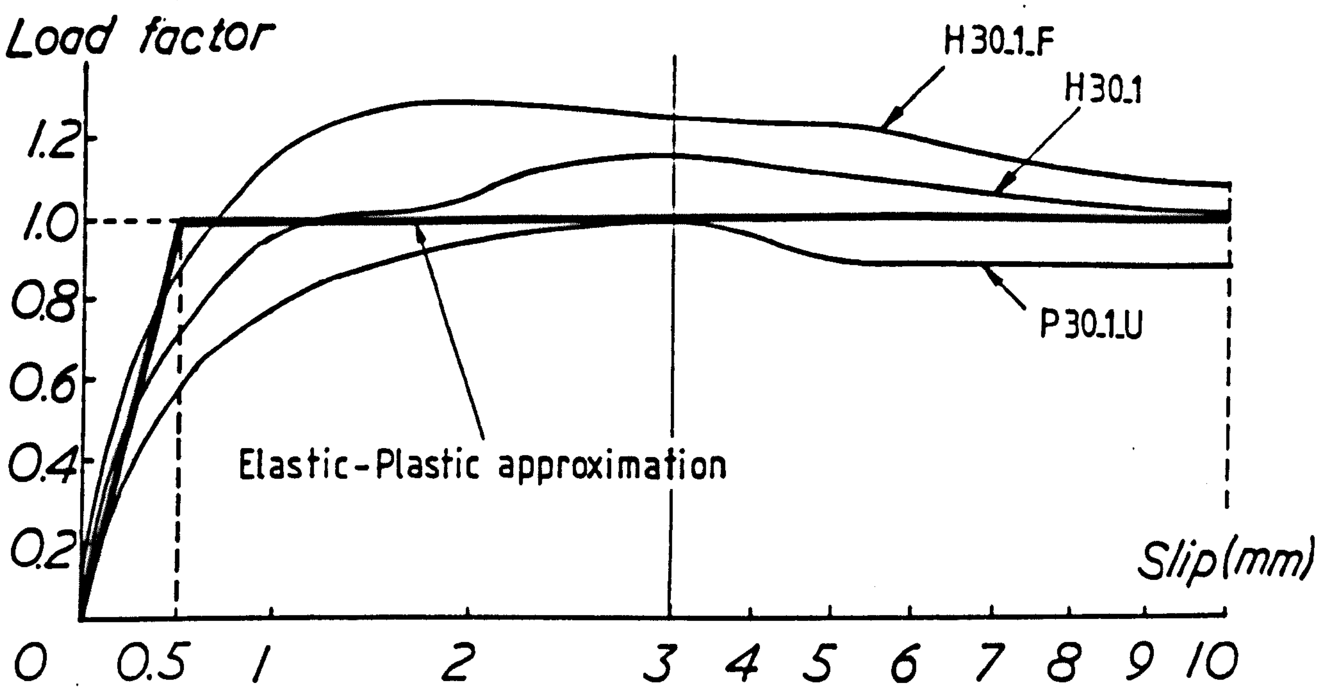


Fig. 7-8, Results of three push out tests by Mottram & Johnson(1990) and the elastic-plastic approximation

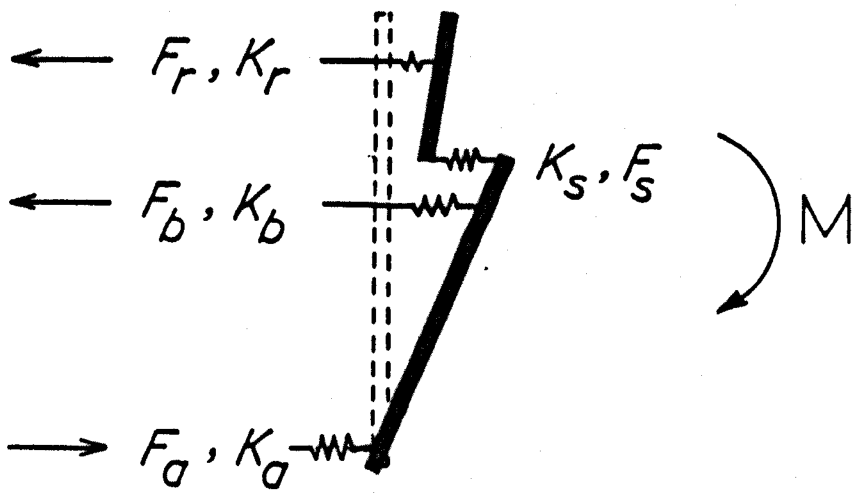


Fig. 7-9, Proposed model including steelwork joint, shear connectors and reinforcement

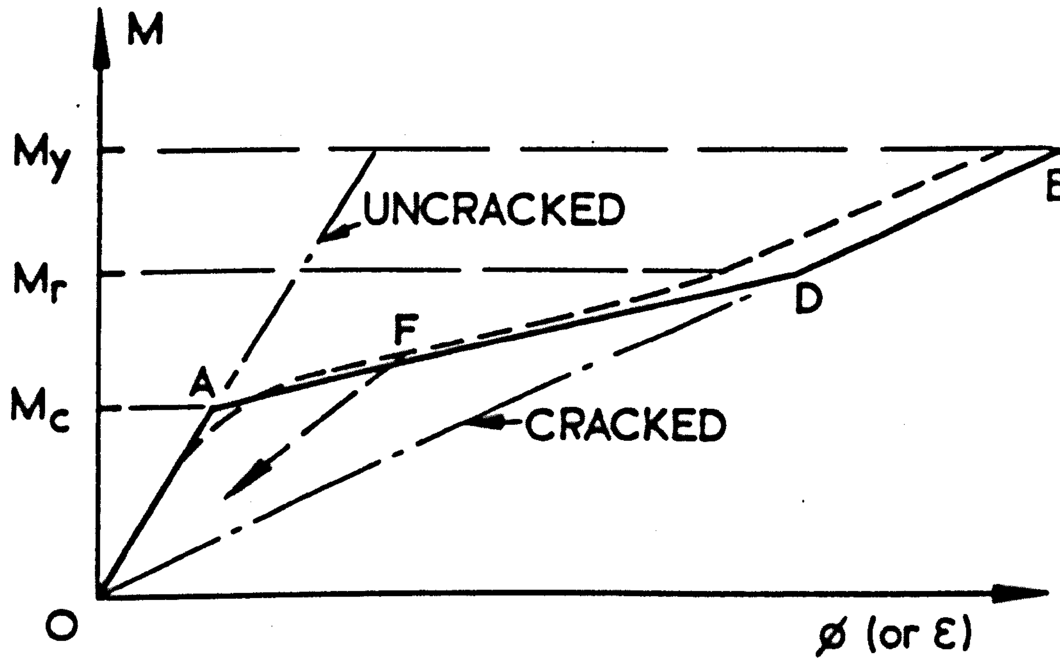


Fig. 7-10, The $M-\phi$ and $M-\varepsilon$ behaviour of a composite section under negative moment (Johnson & Allison(1981))

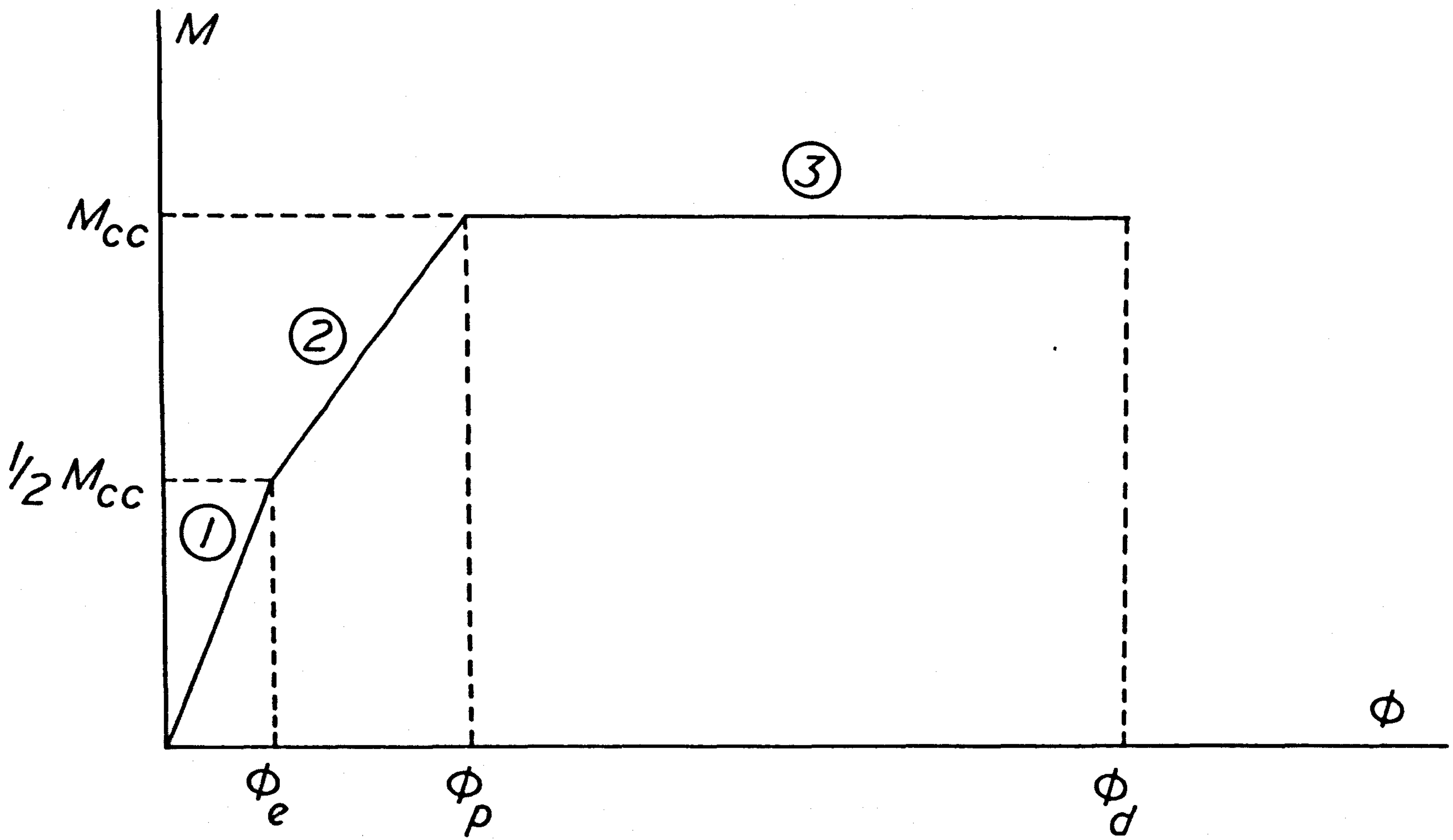


Fig. 7-11, Proposed moment-rotation curve taking uncracked and cracked concrete into account

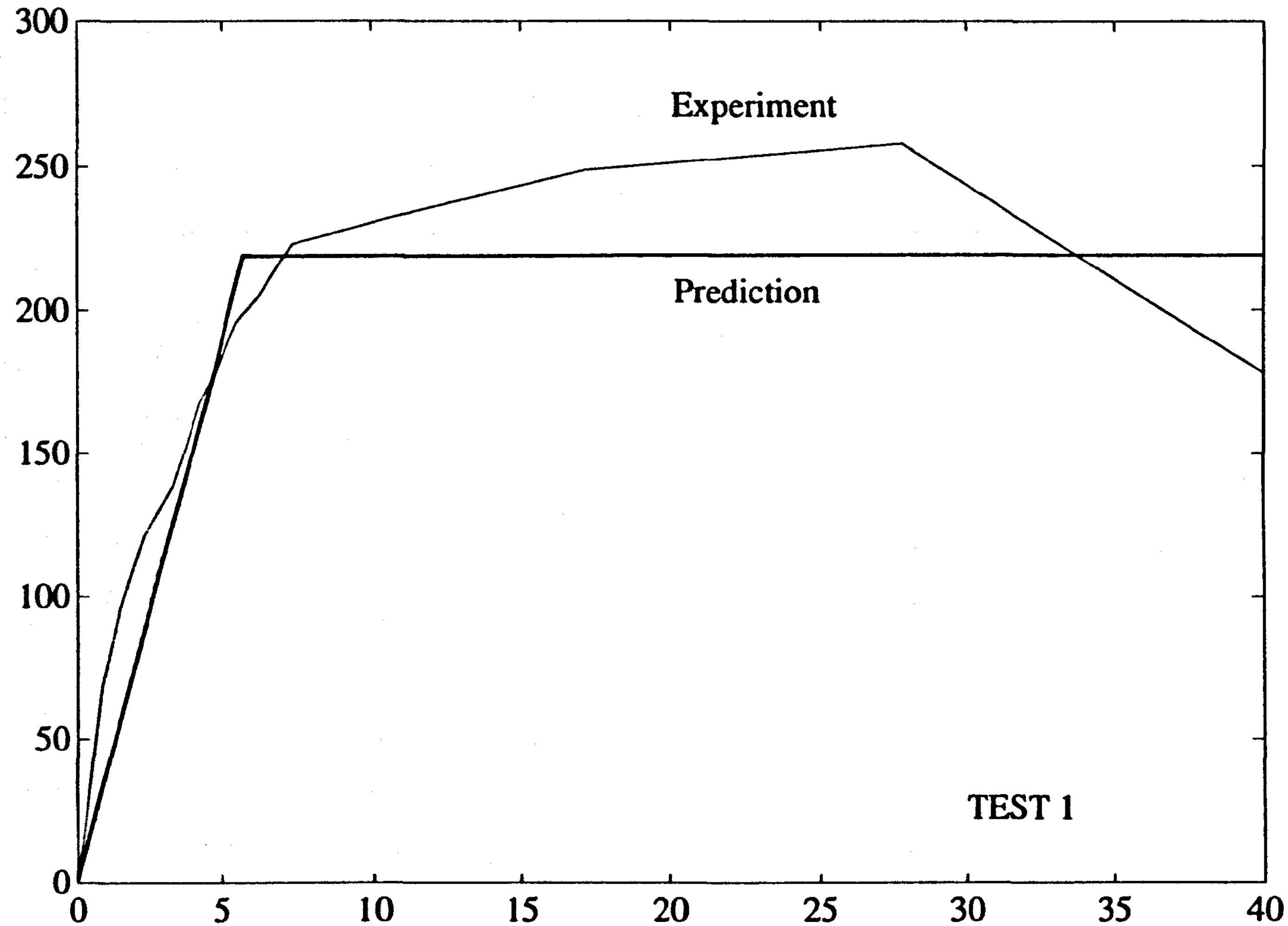


Fig. 7-12, Experimental moment-rotation curve of Test 1 compared to the result of proposed prediction method

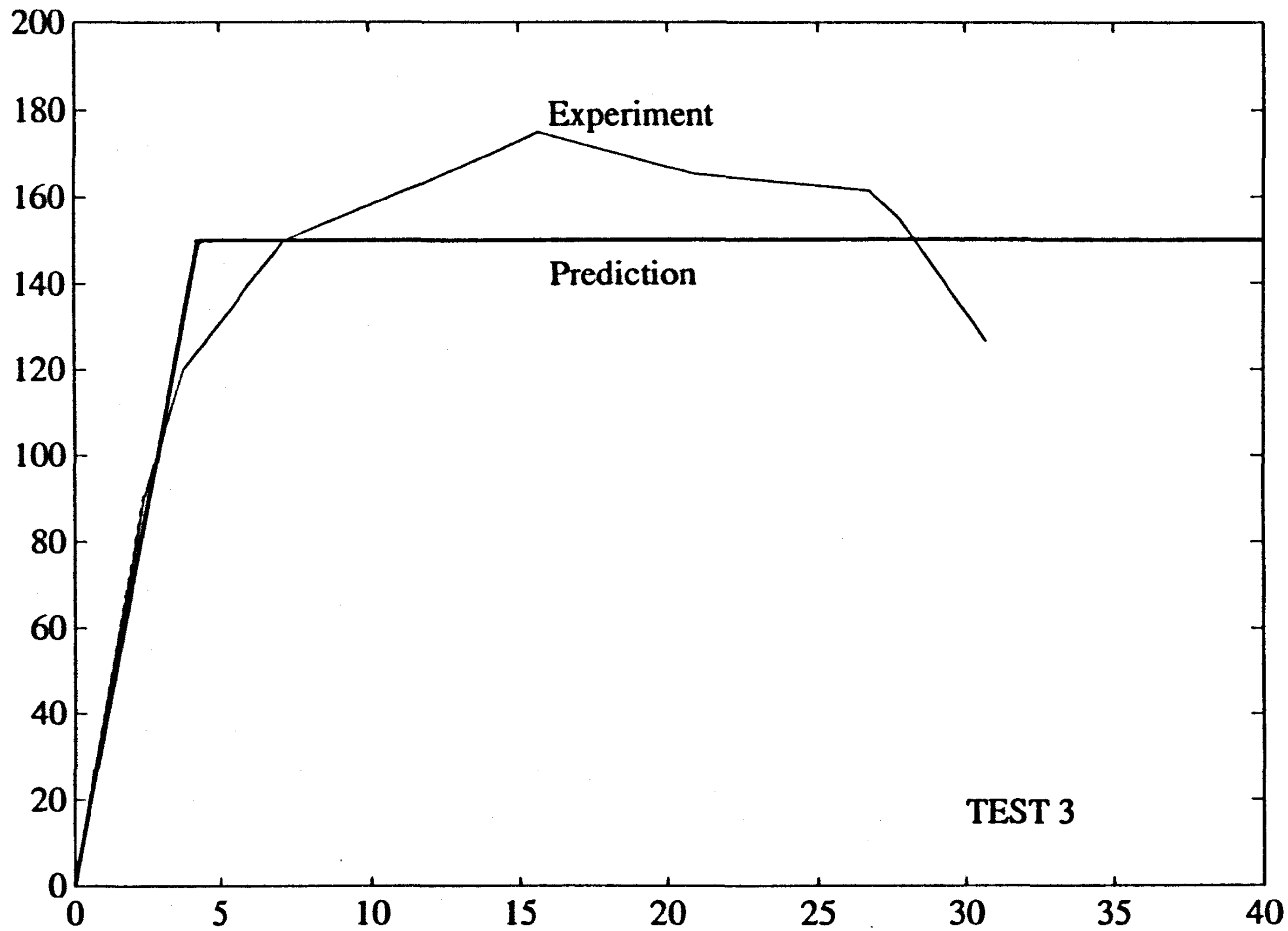


Fig. 7-13, Experimental moment-rotation curve of Test 3 compared to the result of proposed prediction method

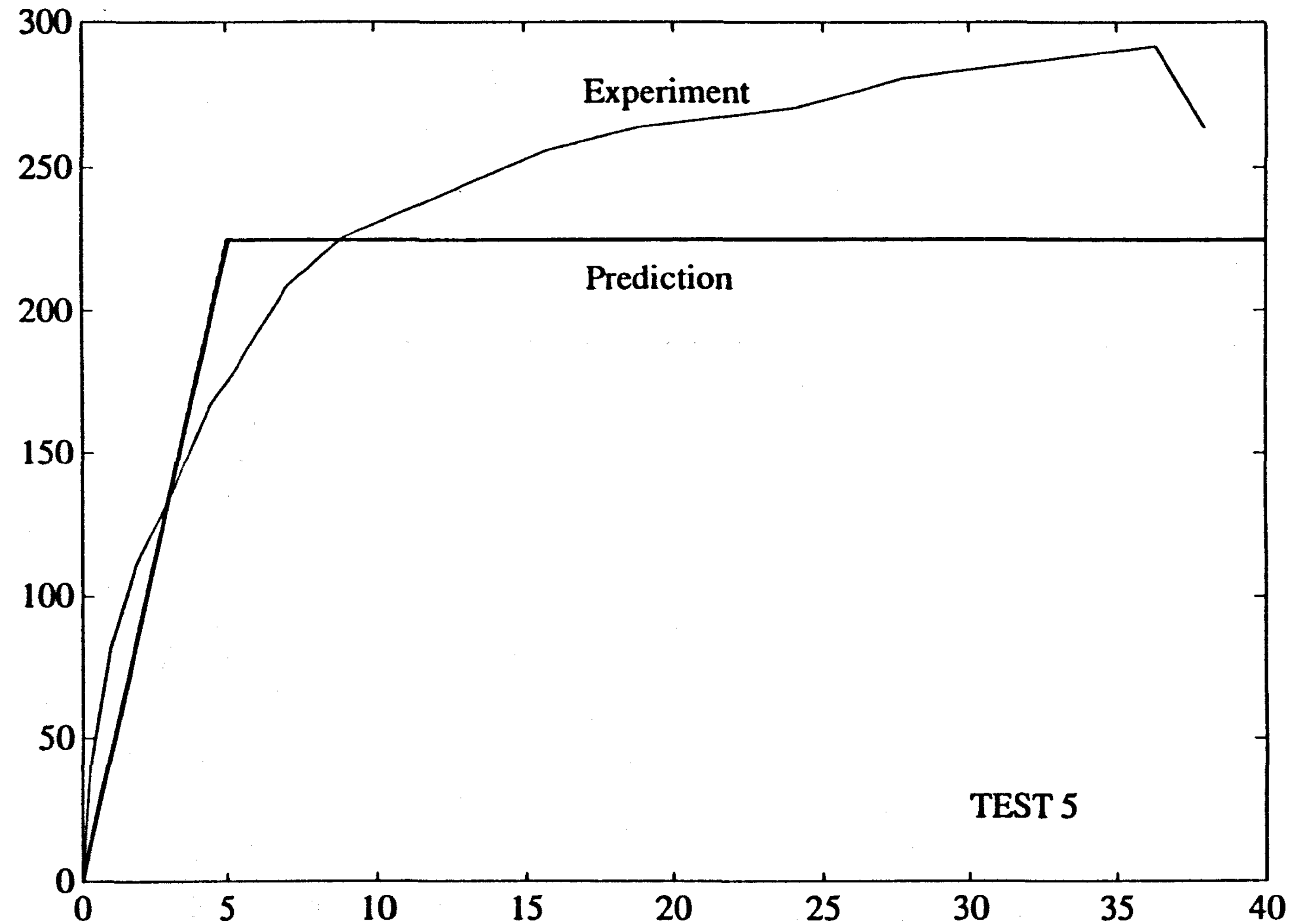


Fig. 7-14, Experimental moment-rotation curve of Test 5 compared to the result of proposed prediction method

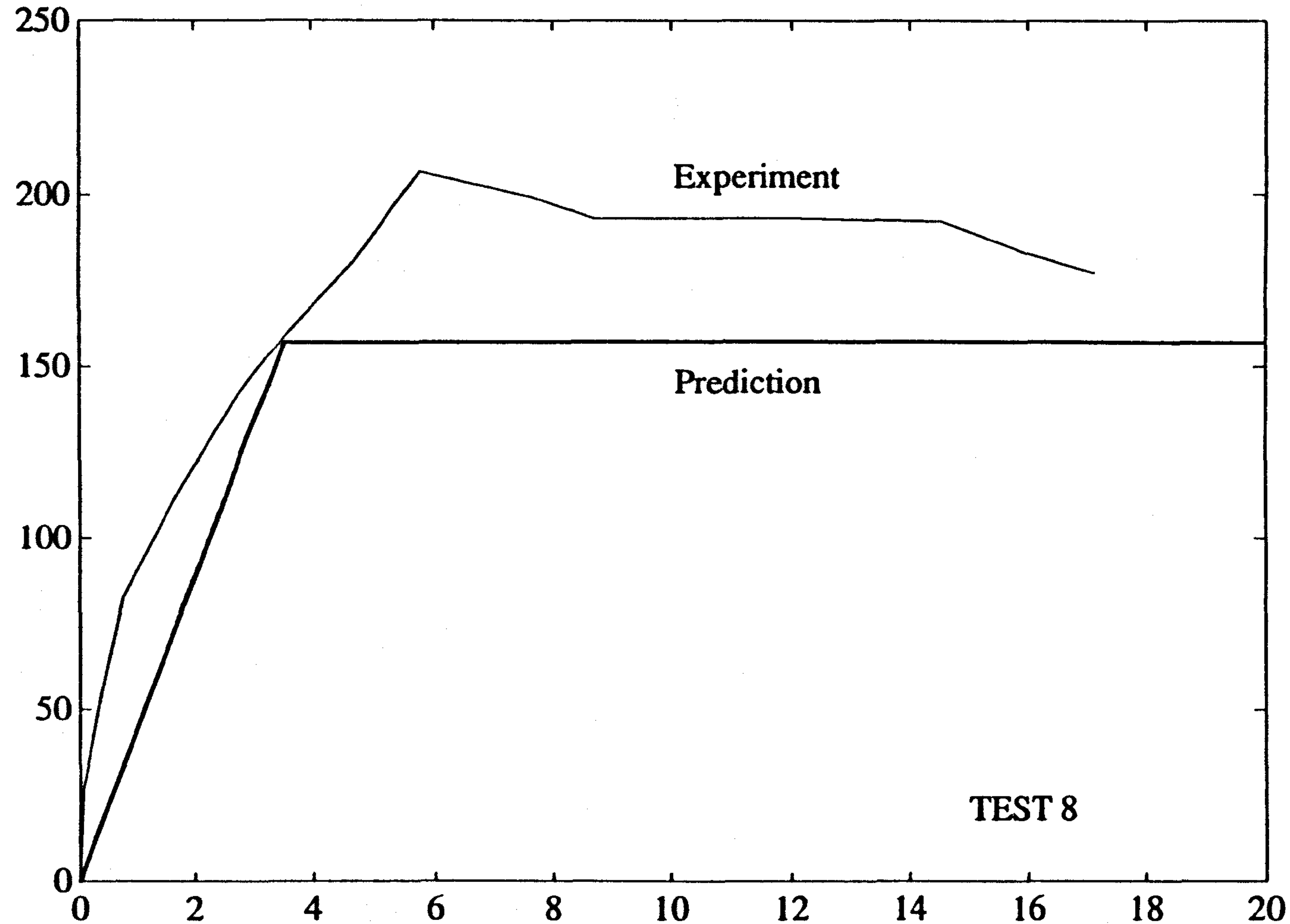


Fig. 7-15, Experimental moment-rotation curve of Test 8 compared to the result of proposed prediction method

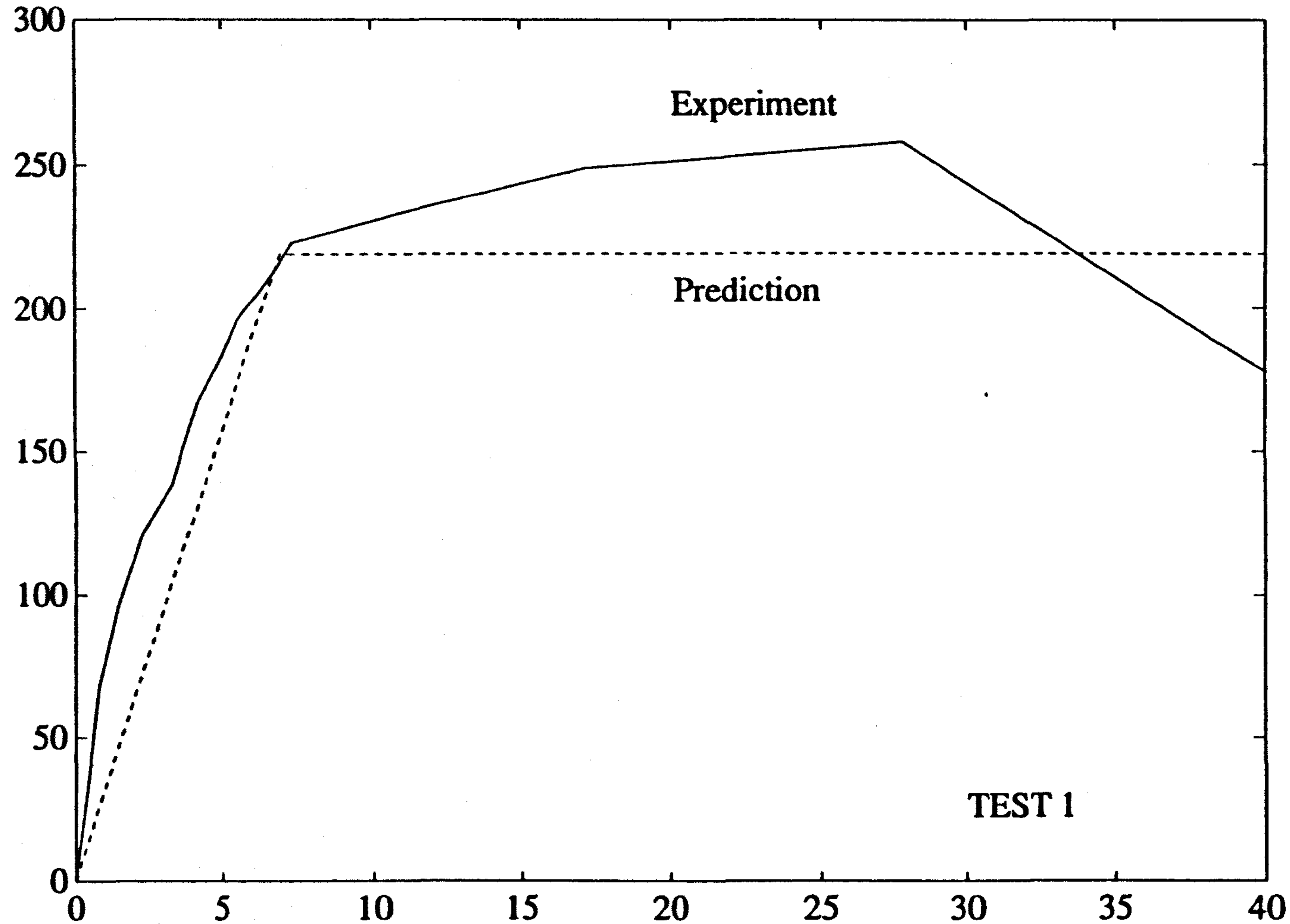


Fig. 7-16, Experimental moment-rotation curve of Test 1 compared to the result of proposed prediction method using EC3 for steelwork joint

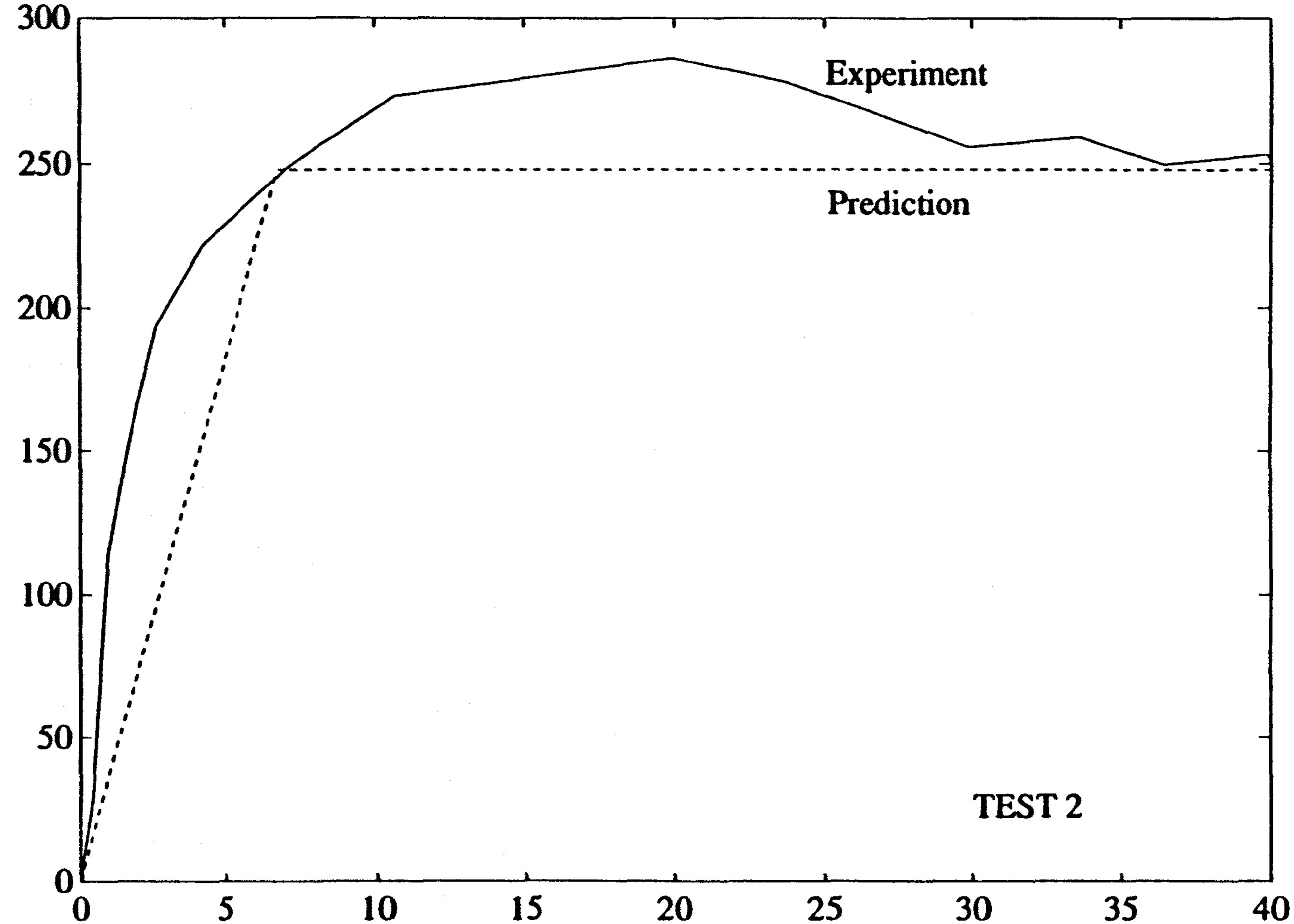


Fig. 7-17, Experimental moment-rotation curve of Test 2 compared to the result of proposed prediction method using EC3 for steelwork joint

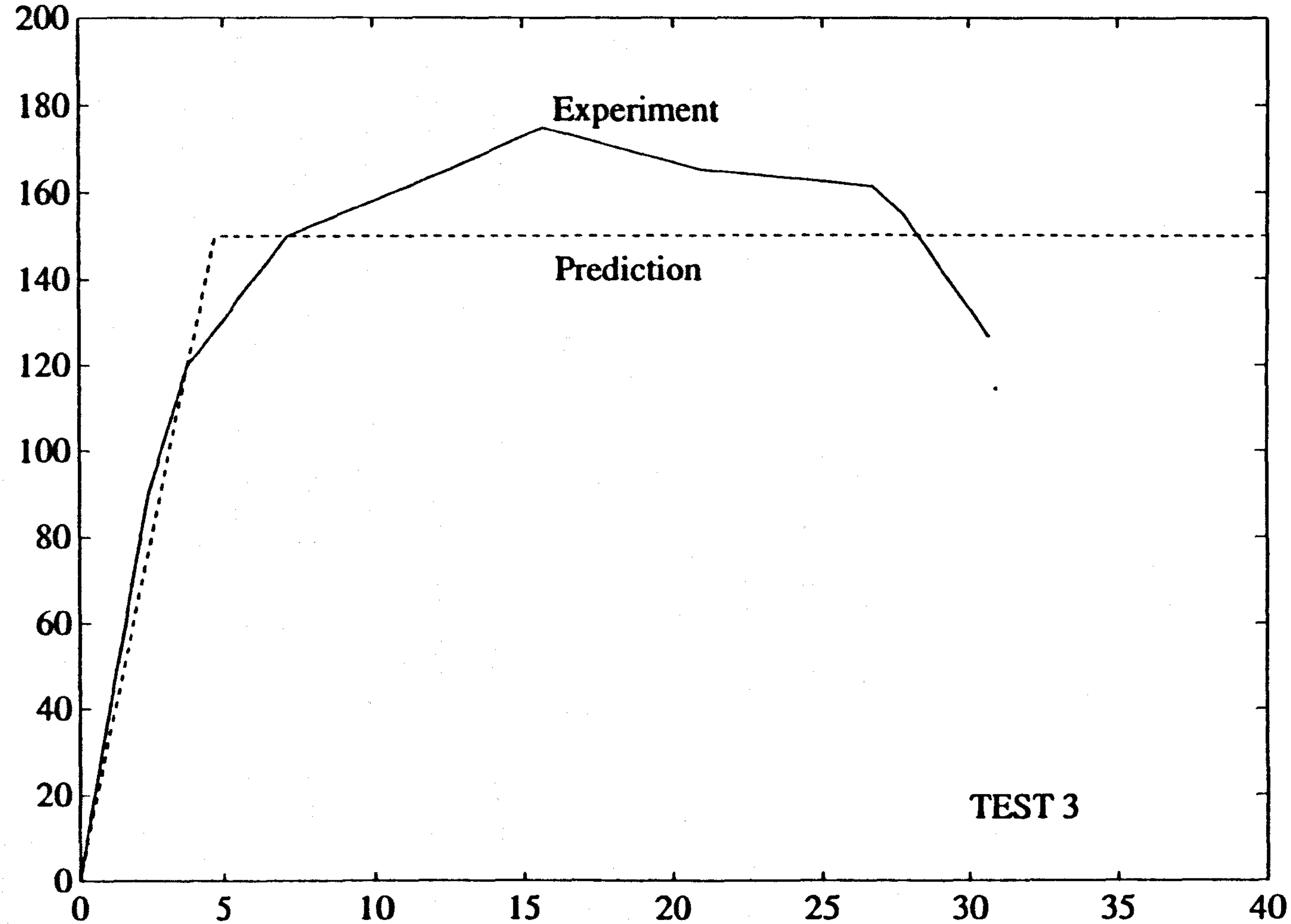


Fig. 7-18, Experimental moment-rotation curve of Test 2 compared to the result of proposed prediction method using EC3 for steelwork joint

Chapter 8

REDISTRIBUTION OF MOMENTS IN COMPOSITE BEAMS

Methods for overall frame design were briefly considered in Chapter 4. This chapter concerns continuous composite beams. The methods of analyis of composite beams are described first, and the background to the subject of redistribution of moments and the plastic design of continuous composite beams is reviewed. The maximum redistribution of moments allowed in composite beams is then investigated. The required rotation of the connection in order to develop a full plastic mechanism is subsequently considered.

In order to investigate the above subjects, a computer program has been developed. This program is described and the results of numerical studies are presented.

For research, a single beam can simulate a beam within a rigidly jointed frame, or a span of a continuous beam, if suitable end conditions are provided. Accordingly, a single span beam has been used for the numerical study of the behaviour of composite beams with semi-rigid end connections.

8-1-Methods of Analysis

In principle, the distribution of bending moments in a continuous beam can be determined by either elastic global analysis or by plastic hinge analysis. These methods are described herein.

8-1-1-Elastic Global Analysis

In elastic global analysis, the composite section may be assumed to be "uncracked" throughout the length of the beam. Alternatively, the "cracked" section may be used for regions on each side of the internal supports, and the "uncracked" section elsewhere. A length equal to 15% of the span is usually taken on each side as the "cracked" region.

Moments are determined for the worst cases of pattern loading:

a) Factored imposed load on two adjacent spans, which generates the maximum negative moment.

b) Factored imposed load on alternate spans with dead load on other spans; this generates the maximum positive moment in the midspan region.

In composite beams, the flexural stiffness of the section in negative bending is usually less than that in midspan, while considering case (a), the negative bending moment at the internal support is usually more than that at midspan. In design therefore the negative bending moment may be redistributed in a way that satisfies equilibrium, taking account of the effects of cracking of concrete, inelastic behaviour of materials, and local buckling of structural steelwork. The shear forces should also be adjusted if necessary. The redistribution of moments in a continuous beam is illustrated in Fig. 8-1.

The permitted redistribution of moments depends on:

1) the moment resistances of the composite connection and composite section in hogging and sagging bending,

2) the rotation capacity of the connection.

The moment capacity of composite connection can be determined from a plastic analysis of the connection. The moment capacity of the composite section in hogging bending is limited by the local buckling of the steel section and the resistance of the reinforcement. The moment capacity of the composite section in sagging bending can be determined from the resistance of concrete slab and steel section.

The rotation capacity of composite connection is dependent on the deformation of steelwork components in compression and tension, deformation of the reinforcement in tension, and deformation of the shear connectors. Available rotation capacity has been measured in the tests described earlier in this thesis.

8-1-2-Plastic Hinge Analysis

Global plastic hinge analysis is based on a collapse mechanism of plastic hinges. The internal beam spans can be treated as conventional three hinge mechanisms if the beam section is Class 1 plastic in both hogging and sagging bending. Fig. 8-2 shows the formation of plastic hinges to develop a collapse mechanism.

The sequence of the formation of plastic hinges in a composite beam with semi-rigid connections depends on the moment distribution along the beam. The shape of the free bending moment diagram and the associated curvature depends on the loading type and arrangement, as well as the maximum moments that can be absorbed by the connections and the beam sections in hogging and sagging bending. Therefore the influential factors in the formation of plastic hinges are:

a) The resistance of the composite beam in hogging and sagging bending.

b) The stiffness and strength of the connection.

A stiffer connection rotates less compared to a more flexible connection at the same moment. Depending on its strength, it may become plastic well before the composite section at midspan. Normally, stiffer connections possess a significant moment resistance. Nonetheless, as described in Chapter 6, a composite connection with lack of ductility in the reinforcement exhibits rigid behaviour while its moment resistance is modest.

Considering a composite beam with two semi-rigid connections, several cases may occur as load increases, depending on the connection strength:

1) Partial strength connection - The connection becomes plastic when neither the beam at support nor at midspan has developed a plastic hinge. Assuming a plateau for $M-\phi$ curve of the connection, the support moment can be kept at its ultimate level, allowing the sagging moment to increase under further load. As long as rotation capacity is available at the joint, the procedure could continue until the maximum sagging moment resistance is attained. Where the first hinge forms at midspan, e.g. under a point load, further load can be applied only when the composite section in sagging bending is Class 1 Plastic. In this case, plasticity extends along the beam in midspan until a plastic hinge forms at the support.

2) Full strength connection - The first plastic hinge may form either at the support or midspan, depending on the stiffness of the beam in hogging and sagging bending, and the stiffness of the connection. For Class 1 sections, rotations may occur until a plastic mechanism is formed.

Codes of practice have adopted several geometric and loading criteria in order to use the plastic hinge analysis. Eurocode 4 permits either rigid-plastic or elastic-plastic methods. The latter includes elastic-perfectly plastic and elasto-plastic methods.

8-1-3-General

The moment resistance and the rotation capacity of composite connections are of particular importance in both elastic global and plastic hinge analysis. It was described earlier that the rotation capacity of composite connections is limited and depends on the ductility of the joints. Therefore, it is important to know what rotation capacity is needed to permit degrees of redistribution commonly used in elastic analysis and the full redistribution assumed in plastic analysis. The connections should be capable of deforming without significant loss of resistance to allow for the redistribution of moments assumed in design to take place.

The aim of this chapter is to provide indications of the required rotation of connections in order to achieve a specified redistribution of moments in composite beams, and also to develop a plastic mechanism.

8-2-Background to Moment Redistribution in Composite Beams

Both steel and composite beams exhibit ductile behaviour due to the elastic-plastic characteristic of materials. In the plastic design of continuous beams, the hinges form where the largest plastic rotations occur. For development of a full plastic mechanism, a certain amount of rotation is required to take place at the hinge while maintaining the predicted moment of resistance. The required rotation of plastically designed members, have been investigated by researchers including Kemp(1986) and Kemp & Dekker(1987).

The suitability of a steel section for plastic design is evaluated by the susceptibility of the section to local buckling. Accordingly, the steel sections have been divided into four categories as explained in Chapter 1. These are:

a) Class 1 Plastic.

b) Class 2 Compact.

c) Class 3 Semi-Compact.

d) Class 4 Slender.

This classification has also been adopted for composite sections with definitions and limits appropriate to composite beams. The composite sections are classified for sagging and hogging bending separately due to the different arrangement of the stress blocks in these regions. The top flange of the beam is assumed fully restraint by the concrete and the action of the connectors. In the sagging region therefore, the top flange (in compression) is not considered liable to buckle; the web still needs to be checked unless (as is often the case) the plastic neutral axis is so high that only a small part of web is in compression. In the hogging region, the bottom flange and a significant part of the beam web are in compression and should be checked against local buckling. The presence of longitudinal reinforcement in hogging region raises the plastic neutral axis, and the section becomes more susceptible to local buckling.

The applicability of plastic theory to composite beams has been investigated by researchers since the 1960's. Barnard & Johnson(1965a)(1965b) and Johnson et al(1965) presented simple methods of predicting the ultimate moment resistance of composite beams based on the simple plastic theory. In these methods, the slip at the steel-concrete interface was neglected. From tests they concluded that the methods were still applicable in the presence of interface slip. Johnson et al(1966) investigated the effect of longitudinal and transverse reinforcement over the supports. The simple plastic theory was proposed for design in which full use could be made of tensile strength of the longitudinal bars.

Because of the need for adequate rotation capacity, Climenhaga(1970) conducted experimental and analytical studies on the effect of local buckling on the moment-rotation characteristics of composite beams. He proposed a set of slenderness rules for unstiffened I-sections to demonstrate suitability for the application of plastic design.

Johnson & Hope-Gill(1976) further examined the applicability of simple plastic theory to continuous composite beams. They used the moment-curvature data from their previous research in a computer analysis to determine the conditions under which continuous composite beams can reach their plastic design loads.

Hope-Gill(1979) extended his work to cover the redistribution of moments in composite beams with non-plastic cross sections. He calculated the redistribution percentage in comparison with elastic uncracked and elastic cracked moments. By plotting these percentages against the vertical loads, criteria for design of compact sections could be concluded. He suggested a redistribution of up to 40% for uncracked analysis, and up to 25% for cracked analysis. For slender sections, assuming the negative design moment as 0.75 of the hogging moment resistance, an amount of 10% redistribution was proposed even if cracked analysis is used. A further 10% was found reasonable when uncracked analysis is carried out, to allow for cracking.

A detailed analysis of cross sections of composite beams in relation to the moment-curvature characteristics was presented by Rotter & Ansourian(1979). They defined a ductility parameter, to be compared with unity to decide whether a section is suitable for plastic design. From tests and numerical results, Ansourian(1984) concluded that for continuous composite beams having a compact steel section, the available rotation capacity in the hogging regions is usually adequate to shed moments to the sagging regions.

Johnson & Fan(1988) conducted a parametric study on redistribution of moments in composite continuous beams with semi-compact cross sections. The aim was to examine the methods given in Eurocode 4 for the design of such beams. They used test data from the post-local-buckling behaviour of cantilevers reported by Climenhaga(1970) to analyse a number of two-span beams. For this study, Fan(1990) developed further a computer program, originally written by Johnson(1982) for a single fixed-end beam, to deal with two-span continuous beams. The effects of residual stress and unpropped construction were studied and found to have little influence on the load capacity of the beam. This was despite other researchers including Rotter & Ansourian(1979) reporting substantial increase in curvature due to residual stress; and decrease in failure load due to unpropped construction (Yam(1981)). Local buckling did not occur until well above service load levels, for both propped and unpropped construction. Johnson and Fan concluded that the method of the draft of EC4 was safe and economical for the ultimate limit state.

Recently, Chen(1992) extended the Fan's work on the buckling of inverted U-frames with Class 3 sections to those with Class 4 sections. The main interest is the behaviour of the bridge beams over internal supports, where local and lateral buckling are the governing factors. The need for lateral restraint in these regions has been studied and suggestions made.

8-3-Computer Program "SRCB"

A computer program, initially written by Johnson(1982) for fixed-end beams, has been developed by the present author to include the effect of semi-rigid joints. The program first derives moment-curvature relationships for the beam's cross section in hogging and sagging bending from geometric data and material properties. An initial value is assumed for the support moment. Load is then applied and the program iterates, changing the support moment, until the slope at midspan is equal to zero.

The following assumptions have been made in the elasto-plastic analysis of the program:

1) Plane sections remain plane after bending.
2) Full shear connection exists and no slip occurs at the steel-concrete interface.
3) Materials obey the simplified stress-strain curves given in 8-3-1.
4) The contribution of the decking and the effect of tension stiffening are ignored.

The procedure followed in the program is elaborated in the following sub-sections.

8-3-1-Material Properties

The material properties are illustrated in Fig. 8-3. The stress-strain curves used for concrete and reinforcement are as given in BS5400(1984). The curve for structural steel is as used by Johnson & Fan(1988). The strain hardening of steel is allowed for by assuming a decreased modulus of elasticity, usually taken as the ratio $E_s/33$. The residual stresses in the steel section are assumed to be as shown in the figure, with a compressive stress of $\sigma_y/2$ at the tips of the flanges.

The partial safety factors have been taken as 1.0 where the connection test results have been examined. For design considerations, γ_m values have been assumed for reinforcement and concrete as those given by EC4. A partial safety factor of 1.0 was assumed for the structural steel.

8-3-2-Moment-Curvature Relationships

The components of the composite section are first divided into slices as shown in Fig. 8-4(a). For propped construction, no initial strain and stress are assumed in the section. For unpropped construction, initial elastic sagging and hogging moments are input, representing load on the steel section at the end of construction. The program calculates the initial strains due to these moments from the corresponding curvature using the following equation:

$$\kappa = \frac{M}{EI} \tag{8.1}$$

The initial strains are added to those due to the residual stresses. In generating the moment-curvature relationships, the initial total strains (not stresses) are assumed to exist in the section. Thus, the resulting moment at each stage includes the initial bending moment due to lack of propping.

Considering hogging bending, the elastic properties of the cracked composite section are calculated. An initial value of curvature is assumed as half of that corresponding to a moment on the composite section which would cause yielding of the bottom flange. A trial position for the neutral axis is guessed, which is slightly above the centroid of the steel section.The corresponding strains in the reinforcement and the steel section are then found. From the stress-strain curves of the materials, the stress in each slice and then the axial force in each slice are calculated.

A small change is then made in the neutral axis depth and new strains are found. Iteration, using the Newton-Raphson method, continues until the neutral axis depth is such that the equilibrium of axial forces in the composite section is satisfied. The moments produced by the axial forces of all slices are added together to give the total moment of the section. This moment and the corresponding curvature generate a point of the moment-curvature curve.

The curvature is then increased by an increment factor, and the above cycle is repeated. The procedure continues until the bottom flange stress (accounting for strain hardening) exceeds 1.3 times the yield stress of the structural steel.

For sagging curvature, the method is as above, except for the following. The initial trial position of the neutral axis is assumed to be at one tenth of the depth of the web below the steel top flange. Forces in the concrete and the steel slices are found from the known strains, and equilibrium of axial forces in the composite section is satisfied by convergence onto the neutral axis depth. The sagging moment and curvature then are found and output.

8-3-3-Analysis of Fixed-End Beam

The loading can be input as either point or distributed load. Unpropped construction is included by introducing the relevant sagging and hogging moments on the steel beam.

The initial hogging length of the beam is assumed as 20% of the span adjacent to the beam ends. The hogging and sagging region is divided into elements, as shown in Fig. 8-4(b).

An initial end moment is assumed as a fraction of that for yield of the bottom flange, usually half of that, and the midspan moment as 70% of this initial end moment. The support moment remains constant, and the true value of midspan moment is found by iteration. The iteration concerns the integration of curvature at each node by the Simpson's rule to find the slope at midspan:

$$\theta_{midspan} = \int_0^{L/2} -\frac{M}{EI}\,dx \tag{8.2}$$

If this slope is not close to zero, the midspan moment is changed and the integration repeated until the midspan slope is virtually zero.

The true bending moments are output. The load factor is found from equilibrium and output together with the number of yielded slices. The percentage of redistribution of moments is calculated by comparing the support moment with the elastic support moment of the fixed-end beam. Next cycle begins after the support moment is increased by an increment factor.

The failure load is assumed to be reached when either:

1) The support or midspan moment exceeds the maximum moment for which moment-curvature data are available. A scheme has been built in the program to decrease the load increment when either of these maximum moments are approached.

2) The number of iterations necessary for convergence on the slope at midspan exceeds a specified value, usually taken as 20.

3) The vertical shear exceeds three times the shear resistance of the elastic part of the web.

4) The number of cycles for the beam collapse exceeds a specified value, taken usually as 30.

8-3-4-Incorporation of Connection Rotation

A subroutine has been developed to incorporate the behaviour of the connections at the supports into the program. The method is based on the unique nature of the joint rotation and the beam slope. A beam with semi-rigid connections at its ends is shown in Fig. 8-5. The slope at midspan is equal to the rotation of the joint plus the integration of curvature along half the span of the beam:

$$\theta_{midspan} = \phi_{con} + \int_{0}^{L/2} \kappa dx \tag{8.3}$$

The moment-rotation characteristic of the connection is input in the form of a multilinear relationship. The rotation corresponding to the support moment at each load level is found from the $M-\phi$ curve of the connection. This value is taken as the beam end rotation and integration is carried out to find the slope at midspan. If this slope is not small enough, iteration continues in the same manner as described in 8-3-3 until it is virtually zero.

In the next cycle, the new support moment is used to find the corresponding rotation of the connection. This procedure continues as long as the rotation of the connection is available.

As the support moment increases, i.e. more load is applied to the beam, the moment-rotation curve of the connection approaches its plateau. Subsequently, the increment factor applied to the support moment is reduced to slow down the loading. This is to distinguish clearly between the formation of the first plastic hinge at the connection, the beam end or at midspan. This is necessary when the hogging moment at the support becomes close to the resistance of either joint or beam, which may also coincide with the attainment of the sagging moment resistance.

The different cases that may occur in the plastic hinge analysis of a composite beam with semi-rigid connections, already described in 8-1-2, have been included in the program. In the case of a partial strength connection, when first hinge forms at the joint, the support moment is thereafter kept constant while the load increases to form the plastic hinge at midspan. Effectively, it is assumed that a horizontal plateau has been reached on the connection $M-\phi$ curve. The load factor and the bending moments at nodes along the beam are found from equilibrium. The nodal curvatures are found and integrated to find the required rotation of the connection for this mechanism.

In the case of a full strength connection, when the first hinge forms in the beam at the support, the program ends because the behaviour of the beam from now onwards depends on the classification of the section in hogging. For the first hinge at midspan, the nodal moments and then curvatures are found and integrated to find the rotation of the connection. The flowchart of the computer program is given in Fig. 8-6.

For propped construction, it is assumed that no rotation occurs in the steel connection in the construction stage, although it is known that in reality some rotation may occur depending on the spacing of the props. For unpropped construction, the initial rotation in the steelwork connection is found by using the beam line method with the initial secant stiffness of the steel joint which is input as data (see Fig. 8-7). The composite connection is then assumed to possess this initial rotation after construction is completed.

The EC4 recommendations for the effective breadth of the concrete slab were included in the program:

sagging region $b_e = 0.175\,L$

hogging region $b_e = 0.125\,L$

The program was also modified so that it could deal with both a solid slab and a composite slab. In the latter case, the depth of the decking is input while it is taken as zero for solid slab.

8-3-5-Verification of Computer Program

To verify the original program, hand calculations were performed to check the equilibrium of the forces calculated by the program as acting on the cross sections and along the beam. The computed ultimate hogging and sagging moment resistances were also checked against the values calculated using the standard formulae given in Appendix B of BS5950:Pt 3.1. The iterations in the program which lead to the compatibility condition of zero slope at midspan were also followed by hand calculation.

Having verified the original program, the subroutine for semi-rigid connections was added and checked to ensure computed connection moment and rotation did lie on the $M-\phi$ curve with the moment at the support. Furthermore, a very stiff connection was assumed and semi-rigid program was run; the results were close to the results of fixed-end beam using the original program. Equilibrium and compatibility conditions were also checked for the beam with semi-rigid connections in a similar manner to previously.

8-3-6-Application of Plastic Analysis to Non-Plastic Sections

Since the program develops plasticity in all slices of the steel section, only composite beams with Class 1 sections should be dealt with by the program. In the analyses in this chapter, it has been assumed that all sections can develop their full plastic moment resistance. This was made in order to study the rotations required for plastic global analysis. However, the tests reported earlier provide some physical justification for the classification of the webs if they are not "plastic".

Considering the connection of Test 1, no sign of local buckling was observed in the Class 3 composite beam section, although the connection was full strength (see Fig. 6-6). This implies the possibility of assuming the beam of Test 1 to be treated at least as Class 2 Compact, which can develop the full plastic moment resistance of the composite section. This confirms the "hole-in-the-web" approach of BS5950:Pt 3.1 for calculation of resistance in such sections. By definition, a Class 2 section does not sustain the plastic moment resistance. However, in Test 1 the applied moment exceeded that of the beam's plastic resistance for significant rotation without any local buckling, indicating Class 1 behaviour.

The composite beam of Test 7 used more reinforcement and had also been classified as Class 3. The whole depth of the beam web was in compression as the plastic neutral axis was in the top flange due to the presence of so much reinforcement. The moment values are as following:

calculated moment resistance of composite connection	264 kNm(R)	277 kNm(R+M)
calculated plastic moment resistance of		
composite beam (accounting for "hole-in-the-web")	271 kNm(R)	281 kNm(R+M)
first sign of local buckling in the beam flange	252 kNm	
maximum experimental moment after significant buckling	298 kNm	

It is seen that in practice the connection is full strength, with a maximum test moment 10% more than the beam moment resistance ($\bar{m}$=1.1 in Fig. 6-6). Although the first sign of local buckling in bottom flange was observed at 252 kNm (which is less than the calculated resistance of the connection or the beam resistance moment), it may be concluded that plastic design calculations are applicable to such a beam and connection because moments higher than the calculated resistances were sustained by the beam with significant rotation.

8-4-Maximum Redistribution of Moments

Both BS5950:Pt 3.1 and Eurocode 4 give two approaches for the elastic global analysis, namely gross section method and cracked section method. They permit redistribution of support moments, depending on the method used and the classification of the section at each internal support. The permitted redistribution of support moments in BS5950:Pt 3.1 and EC4 are compared in Table 8-1. It should be noted that these codes assume that the bare steel connection is fully rigid.

In BS5950:Pt 3.1, 10% less redistribution is permitted if the cracked section method is used. In EC4, the same difference is adopted for beams with negative regions in Class 3 or 4. For Class 1 or 2 sections, it gives a difference of 15%. This is because the difference between support moments given by uncracked and cracked analysis increases with the ratio of depth of slab to depth of steel section. This ratio is generally higher for sections in Class 1 or Class 2 (Anderson & Johnson(1992)).

Interim suggestions have been made on the possible maximum redistribution of moments in the beams with semi-rigid connections (Lawson(1991)), but none of the codes has yet given criteria for the design of such members. As mentioned earlier, EC4 is still lacking instructions on semi-rigid composite connections generally, and consequently, no indication of the effect of such connections has been given in the code. The studies reported here aim to obtain some indication of the amount of redistribution that may occur in a composite beam with semi-rigid connections.

8-4-1-Pilot Study

In order to gain an overall view of the subject of redistribution of moments in semi-rigid composite beams, a pilot study was made.

Consider the beam of Fig. 8-8(a) with a span of L under uniformly distributed load w. The redistribution ratio r is defined as the ratio of moment that has been shed from the support to the midspan, to the rigid elastic moment at the support. Therefore the moment at the beam end following redistribution is:

$$M_h = (1-r)\,M_e = r'\,M_e \tag{8.4}$$

where M_h and M_e are the support moment after and before redistribution as shown in Fig.8-8(b). The following equation can be written:

$$M_{ps} + r'\,\frac{wL^2}{12} = \frac{wL^2}{8} \tag{8.5}$$

which gives the plastic sagging moment resistance as:

$$M_{ps} = (1-\tfrac{2}{3}r')\,\frac{wL^2}{8} = (\tfrac{1}{3}+\tfrac{2}{3}r)\,\frac{wl^2}{8} \tag{8.6}$$

The value of r' can then be found in terms of the plastic sagging moment as:

$$r' = 1.5 - \frac{12}{wL^2}\,M_{ps} \tag{8.7}$$

For a known value of sagging moment resistance and a given value of r' (or r), the applied uniformly distributed load is:

$$w = 12\frac{M_{ps}}{(1.5-r')L^2} = 12\frac{M_{ps}}{(0.5+r)L^2} \tag{8.8}$$

The composite beams similar to those used in the experimental work were chosen as test results of the end connections were then available. The beam of Test 1 was considered, assuming propped construction with the connection stiffness as obtained in the test and the following data:

steel beam	305x165UB40, with span = 9.0 m,
reinforcement at support	905 mm^2, with $f_y = 460\ N/mm^2$ and $\gamma_m = 1.15$,
concrete	Grade 30, with $f_{cu} = 30\ N/mm^2$ and $\gamma_m = 1.50$,
composite joint	end plate, with $M_{pc} = 219\ kNm$ from Table 6-1.

Assuming a redistribution percentage of 40% (r=0.4, r'=0.6), the load w which caused the resistance of the connection to be attained was calculated as 54 kN/m. Using Eqn. (8.6), the corresponding sagging moment was 329 kNm. To sustain this moment with the chosen section, a strength f_{ys} for the steel section would need to be 290 N/mm^2 which is in the range of Grade 43 steel. For these limiting hogging and sagging conditions to be reached, the program calculated a connection rotation of 27 mrad, which is comparable with the rotation of 28.0 mrad achieved at the maximum moment in Test 1.

To examine the beam further, different grades of steel were taken and the beam was analysed to determine the connection rotation and degree of redistribution required to attain a full plastic beam mechanism. The connection resistance moment could still be kept the same, since the weaker components in Test 1 were the column flange and reinforcement, i.e. the connection moment resistance was not dependent on the beam's yield strength. The results are tabulated in Table 8-2. The analysis of the beam with

Grade 43 steel, for which test results are available, gives a rotation of 22.9 mrad for the attainment of full plastic mechanism (Fig. 8-8(c)). The rotation achieved at the maximum moment in Test 1 was 28.0 mrad which is sufficient compared to the required value of 22.9 mrad. Thus the permitted redistribution for plastic sections given in BS5950:Pt 3.1 as 40% is safe for this example.

This last procedure was carried out for the deeper beam of Test 10 (457x152UB52) with a span of 11.5 m and the connection moment of 328 kNm as given in Table 6-1. The material properties of concrete and reinforcing bars were the same as given above. The results are tabulated in Table 8-3. It is seen that the rotation capacity of the connection in Test 10 is significantly less than the rotation required in the plastic analysis of composite beam with Grade 43 steel (as used in the test). Much less redistribution would then have to be allowed, unless the ductility of the connection could be increased e.g. by adjusting the amount of reinforcement.

From comparison between Tables 8-2 and 8-3 it follows that greater redistribution of support moments is needed in a deeper beam to attain the plastic mechanism. This is associated with a modest need for extra rotation capacity of the connections. It is also concluded that higher grade of steel results in more redistribution of moments being required but a considerable increase in rotation capacity will also be necessary.

8-4-2-Effect of Reinforcement on Redistribution of Moments

For a beam with constant sagging moment resistance, the support moment can vary according to the amount of reinforcement provided. The greater percentage of reinforcement results in the greater support moment which in turn increases the load carrying capacity of the beam. For more reinforcement and a given value of loading, there is less need for the redistribution of moments, since the support section is capable of resisting more moment. On the other hand, when less reinforcement is provided, more redistribution is needed. For different amounts of reinforcement therefore, different values of maximum moment redistribution can be defined.

As explained in Chapter 6, in general the more reinforced connections possess greater rotation capacity. This capacity is necessary to provide moment redistribution. Therefore, the more reinforced sections in the hogging region allow more redistribution to take place, although, as mentioned above, there is less need for moment redistribution in beams with such sections.

When uncracked analysis is used, both BS5950:Pt 3 and EC4 permit up to 40% redistribution for Class 1, reducing to 10% for Class 4 section. The values of support moment after redistribution can be found for these permitted percentages. The following equation for hogging and sagging moment exists when the plastic moment at midspan is reached:

$$M_h + M_{ps} = \frac{wL^2}{8} \tag{8.9}$$

For a certain amount of moment redistribution the values of sagging and hogging moments can be derived from Eqns. (8.6) and (8.9) as:

$$\begin{aligned} M_{ps} &= (1+2r)\frac{wL^2}{24} \\ M_h &= (1-r)\frac{wL^2}{12} = 2\left(\frac{1-r}{1+2r}\right)M_{ps} \end{aligned} \tag{8.10}$$

The support moment then can be found for different amounts of redistribution:

Class	Redistribution %	Support Moment
1	40	0.667 M_{ps}
2	30	0.875 M_{ps}
3	20	1.143 M_{ps}
4	10	1.500 M_{ps}

When the amount of reinforcement at the support increases, the classification of the section may change from Class 1 to Class 2 or 3. This is due to shift in the position of plastic neutral axis which results in more depth of web being in compression (Class 4 does not occur usually unless a very high amount of reinforcement is provided with the steel section itself being Slender). The greater resistance of the more reinforced section tends to offset the greater support moment due to less redistribution. However, as plastic resistance for a Class 3 section necessitates use of an effective web, there is also a tendency for the resistance to reduce with increased reinforcement, or at least for further reinforcement to be less effective.

The above discussions and those in Chapter 6 regarding the optimum amount of reinforcement for composite connections imply that a proper selection of reinforcement should be made for an efficient design. This part of the study concerns the connection rotations required for a full plastic mechanism of the beam when the amount of reinforcement in the connection is varied. The moment-rotation curves obtained from Tests 1, 3 and 7 in which 0.5%, 1.0% and 1.5% reinforcement was used respectively, were used in the analyses. The maximum test values of connection moment are those from Table 5-2 which include 4 kNm to account for self weight of the beam. Accordingly, 4kNm was added to the applied test moments for the connection data. Propped construction with the following data has been assumed:

Steel beam	305x165UB40, Grade 43 with f_y=275 N/mm^2.
Reinforcement	4, 8 and 12 T12 bars, f_{yr}=460 N/mm^2.
Concrete	Grade 40, f_{cu}=40 N/mm^2.

The results of the analysis of three beams with the variable reinforcement are tabulated in Table 8-4. As seen, the more reinforced section in hogging region needs less rotation at the end connection for a plastic mechanism. It will be recalled that the test results show that more reinforced connections possessed greater rotation capacity, and those with less reinforcement exhibited a less-ductile behaviour with less rotation capacity. It is therefore concluded that to adjust the reinforcing material to provide a rotation capacity which matches that required from the joint, an optimum amount of reinforcement should be used. For convenience in design, the author suggests an amount of 1% reinforcement with respect to the area of concrete slab (above decking) within an effective breadth as given in EC4 and BS5950:Pt 3.

8-4-3-Redistribution of Moments in Composite Beams with 1% Reinforcement

The above discussions show that the subject of moment redistribution is interrelated to the rotation capacity of the connections. In this section, a number of composite beams comprising British steel sections in the practical range of span and depth are analysed. Both Grade 43 and 50 are assumed. The material properties and safety factors are as those used in 8-4-1. The behaviour of the end connections is taken as that given in Fig. 8-9. The effective breadths in hogging bending and the subsequent amounts of reinforcement used in the analyses are given in Table 8-5.

In these analyses, both propped and unpropped conditions have been considered. The construction load for unpropped situation has been taken as 10 kN/m. No initial load on the beam has been assumed for propped construction. For unpropped construction, the secant stiffness of the steel end plate connection at two-thirds of its maximum resistance obtained in Test 9 was first taken. This stiffness is for a beam depth of 305 mm. For other steel sections the following equation was used to account for variable beam depth:

$$C_D = C \left(\frac{D}{305} \right)$$

where C is the secant stiffness of the steel joint with a 305 mm deep section and C_D is that for a section with a depth of D. To correct this stiffness for Grade 50 steel, the following expression was assumed which had been concluded from averaging a number of available results of connection tests:

$$C_{DG} = 1.136\, C_D$$

The values taken for secant stiffness of steel connection are tabulated in Table 8-6.

First, a percentage of redistribution is assumed. The support moment resistance is then found from Eqn. (8.10) using the sagging plastic moment resistance. The stiffness and the moment resistance of the joint are then both known. Analysis can therefore be performed and the required rotation for development of a plastic mechanism computed.

Initially, the maximum percentage of redistribution permitted in the codes (40%) is examined. The results are given in Table 8-7 for spans of 6.0, 9.0 and 12.0 m.

Then 50% redistribution is assumed which results in a support moment equal to half of that at midspan (Eqn. (8.10)). The results are given in Table 8-8.

Comparing Tables 8-7 and 8-8, the required rotations for the assumed redistributions are less in the propped analysis. It should be noted that in these tables, the values of w for unpropped construction are the applied loads to the beam after the attainment of composite action. From Tables 8-7 and 8-8 the following conclusions can be drawn for composite beams with 1% reinforcement in hogging region and comprising end plate connections at the ends:

a) Generally, more rotation is required for development of plastic mechanism in the beams with smaller steel sections for the same span.

b) Generally, greater spans require more rotation capacity for the end connections.

c) Grade of steel does not have a direct effect on the required rotation of the connection, because change in the steel grade alters simultaneously the moment resistance of composite beam in sagging and hogging bending, as well as the moment resistance of the composite connection which was taken as proportional to the sagging moment in this study.

d) More redistribution of moments necessitates greater rotation capacities for the connections.

e) For 40% redistribution and propped construction, the values of required rotation for the joints are in the range of available test data for connections comprising 1% reinforcement (see Chapter 6).

f) For 50% redistribution and propped construction, a rotation capacity of up to 35 mrad is required of the joints. It is not always possible to achieve this unless the amount of reinforcement is increased. In that case, less redistribution will be required, since the connection and the beam at the support are capable of absorbing more moment. Therefore, assuming more redistribution than 40% is not advisable.

Finally, it should be remembered that it is possible to reduce the rotation requirements by not utilising the whole of the sagging resistance of the beam.

8-5-Required Rotation of Composite Connections

In this section the rotation that occurs in the connection to develop a full plastic mechanism is considered, based on varying connection resistance ($\overline{m} < 1.0$) and on varying stiffness.

When load is applied to a composite beam with semi-rigid connections at its ends, elastic curvatures take place along the beam in sagging and hogging regions associated with rotation at the end joints. At each load level, the compatibility condition implies that the connection rotation is dependent on the deflected shape of the beam.

As more load is applied, plastic deformations also take place which are added to the elastic ones. The plastic mechanism will be reached when yielding of the midspan and support sections is complete.

A plastic hinge at the support may form either in the joint or the beam under hogging bending. The rotation of the joint at this stage is still dependent on the deformed shape of the beam which can be found by integration of curvatures, satisfying the compatibility condition. This procedure was explained in section 8-2 and has been followed in the program to study the subject of required rotation, as described in 8-5-1 below. In section 8-5-2, a formula will be proposed for calculation of required rotation capacity based on the above explanation.

8-5-1-Initial Studies

In order to examine the required rotation of the joints the following study was undertaken. Various beam sections and spans, and variable connection behaviour , were employed. Propped analysis was carried out with the following data:

Steel grade	Grade 43 with f_y=275 N/mm^2.
Reinforcement	1% of the area of concrete above decking within the effective breadth, with f_{yr}=460 N/mm^2 and γ_m=1.15.
Concrete	Grade 30 with f_{cu}=30 N/mm^2 and γ_m=1.5.

A "standard procedure" was chosen. The connection data was given as the rigid, full-strength boundary in EC4 for composite joints in braced frames (Fig. 8-10(a)). Initially, the hogging resistance of the beam was taken as the ultimate moment of the connection and a distributed load was applied. The first hinge occurred at midspan and the corresponding rotation of the connection was found. Then the connection data was given using the same trilinear curve but reducing the plateau to fractions of the hogging resistance moment of the beam. In most of these analyses, the first hinge formed at the supports and then load increased until the midspan hinge formed. The required rotation to develop the mechanism was found for each fraction of the beam's resistance.

Sample results of the analyses are given in Table 8-9. The tabulated values are for situations where first plastic hinge formed at the connection and was then followed by a plastic hinge at midspan. The following can be concluded from this numerical study:

a) The required rotation increases non-linearly as the resistance of the connection decreases.

b) As expected, the load factor for beam collapse increases linearly as the connection strength increases. Examples are given in Fig. 8-11 for 11.5 m span.

c) The required rotation is generally more for larger spans.

d) The required rotation in most cases is less for deeper sections with the same span. Hence conservative design of composite beams results in less restrictive requirement for rotation capacity of connections.

In order to examine the effect of variable connection stiffness, the beam sections used in 8-4-3 were chosen and propped analysis was carried out. In the analyses, the initial rigid boundary for composite connections in braced frames was taken from EC4. Then the trilinear curve was altered so that the two characteristic $\bar{\phi}$ values (0.125 and 0.2) were multiplied by factor β ranging from 1.0 to 4.0, as shown in Fig. 8-10(b). For each curve the same procedure of decreasing connection strength to fractions of the beam resistance (Fig. 8-10(a)) was followed.

In assessing the values of rotation required to develop a mechanism in the beam, it was found that this rotation was unique for the identical ultimate resistances of the connection, regardless of the moment-rotation path that the connection had followed. This achievement seemed rather surprising, so it was examined further by using the beam line method which confirmed the computer analysis. A sample check is given in Appendix H.

To avoid repetitious tables, only the end rotations resulted from analysis of the two beam sections used in the author's tests are given in Table 8-10. It is seen from the table that provided the first hinge forms in the connection, the flexibility of connection does not influence the rotation needed for plastic mechanism, rather its resistance determines the required rotation. This conclusion results in simplifying the search for criteria for the required rotation capacity of partial strength connections in composite frames. As a consequence, the calculation of required rotation can be based on the connection resistance proportional to the moment resistance of the connected beam, either in hogging or sagging bending.

The flexibility of connections may result in the formation of plastic hinge in midspan before the attainment of maximum resistance of the joint. This is shown in Table 8-10 for a very flexible connection (β=4.0).

8-5-2-Derivation of Formulae for Required Rotation

The rotation that takes place at the joint is the resultant of the elastic and plastic rotations along the beam. It is assumed that the connection and the midspan section become plastic not the beam section at the support, i.e. partial strength connections are assumed.

8-5-2-1-Elastic Rotation

This is determined by standard theory, as now shown.

The elastic rotation of the beam end under a distributed load w with a support moment of M_j can be written as (see Fig. 8-12(a)):

$$\theta_e = \frac{wL^3}{24EI} - \frac{M_j L}{2EI} \tag{8.11}$$

Assuming the ratio of support moment to the maximum sagging moment is k_1, for yielding at midspan:

$$M_j = k_1 M_{ys}$$

Also:

$$M_j + M_{ys} = \frac{wL^2}{8}$$

The elastic rotation of the connection will then be:

$$\theta_e = (1+k_1)\, M_{ys} \frac{L}{3EI} - k_1\, M_{ys} \frac{L}{2EI}$$

which gives the following equation for the elastic rotation of the beam end:

$$\theta_e = \frac{M_{ys} L}{3EI} (1-0.5k_1) \tag{8.12}$$

8-5-2-2-Plastic Rotation

Initially, formulae are presented following Gibbons(1992).

The plastic rotation taking place at the beam end can be derived from the length of the yielded zone in the beam under sagging bending (see Fig. 8-12(b)):

$$M_{ys} = M_{ps} - \frac{wx^2}{2}$$

$$l_y = [2\frac{(M_{ps}-M_{ys})}{w}]^{1/2}$$

Defining s as the shape factor of the composite section in sagging bending:

$$s = \frac{M_{ps}}{M_{ys}}$$

and taking the plastic sagging moment resistance as:

$$M_{ps} = \frac{wL^2}{8(1+k)}$$

where $k = \frac{M_j}{M_{ps}}$ to express the joint moment with respect to the plastic moment resistance in sagging bending. The yielded length of the sagging region will be written as:

$$l_y = \frac{L}{2}[\frac{s-1}{s(1+k)}]^{1/2} \tag{8.13}$$

From Fig. 8-12(b) the plastic rotation along the length l_y can be written in terms of strain developed in the beam's bottom flange at the midspan, and at the distance l_y from the midspan:

$$\theta_p = \frac{1}{2}(\varepsilon_{midspan} - \varepsilon_y)\frac{l_y}{D^T} \tag{8.14}$$

where D' is the distance of plastic neutral axis from beneath the bottom flange of the beam. The strain condition at midspan depends on the initial strain due to propped or unpropped construction. Considering propped construction, the initial strains are dependent on the spacing of props, e.g. propping at midspan or two-thirds etc. The author assumes that no initial strain exists in the sections along the sagging region. This is in contrast to Gibbons(1992) who assumed the beam was propped only at midspan. In unpropped construction, the strain at the section is the sum of initial and subsequent strains.

The strains at the midspan section are calculated in Appendix G of this thesis with simplifications suggested by Gibbons(1992). He takes a value of 1.3 as the shape factor and assumes that the neutral axis in the midspan section is at the bottom face of decking in elastic phase and at the top face of decking in plastic phase. For the cases considered in Appendix G, the author shows that a strain of about $4.5\varepsilon_y$ in fully-propped and $6.0\varepsilon_y$ in unpropped construction takes place in the beam's bottom flange at midspan in order to develop at least 95% of the plastic sagging resistance.

8-5-2-3-Required Rotation

With the simplifications made in Appendix G, the elastic and plastic rotations from Eqns. (8.12) and (8.14) can be written for propped construction as:

$$\theta_e = 0.33 \frac{f_y L}{ED} (1-0.65k) \tag{8.15}$$

$$\theta_p = 0.38 \frac{f_y L}{ED} (1+k)^{-1/2} \tag{8.16}$$

where $k = \frac{M_j}{M_{ps}} = \frac{1}{1.3} k_1$ is used to express the joint moment with respect to the plastic moment resistance in midspan.

The required rotation of the connection for compatibility is then the sum of the above rotations:

$$\phi = \frac{f_y L}{ED}[0.33-0.21k+0.38\,(1+k)^{-1/2}] \tag{8.17}$$

Since the simplifications in Appendix G are based on the beams of Grade 43 steel used in the author's tests, a factor multiplies the rotation obtained from Eqn. (8.17) to correct it for Grade 50 steel:

$$\phi = 0.88\,\frac{f_y L}{ED}[0.33-0.21k+0.38\,(1+k)^{-1/2}] \tag{8.18}$$

Gibbons has derived similar equations for beams with other types of loading. He has given the following equation for uniformly distributed load:

$$\phi = \alpha\,\frac{f_y L}{ED}\,[0.33-0.21k+0.22\Omega(1+k)^{-1/2}] \tag{8.19}$$

where factors α and Ω are assumed as unity for propped construction. For unpropped construction Ω is taken as 2.5 and α as:

$$\alpha = \frac{live\ \ load}{dead\ \ load + live\ \ load}$$

The loads are the factored values. It should be noted that in his formule, props are assumed to be located at the beam midspan which results in a hogging-type strain block at the midspan section under construction loads. He simplified the equations for various type of loading into a single expression for the required rotation of the connection:

$$\phi = \frac{f_y L}{ED}\,(0.7\mu-0.2k) \tag{8.20}$$

in which α has been approximated to 0.7, and the values of μ are defined as follows:

	Propped	Unpropped
Two point load	1.0	1.25
Uniform load	0.8	0.9
Single point load	0.5	0.5

The author disagrees with the application of α to the end rotation to take account of unpropped construction for two reasons:

1) The total rotation which includes elastic and plastic rotations should not be multiplied by a single factor. The state of the end rotation at the completion of construction phase can separately be calculated (which is normally elastic) and considered as the initial rotation at the service phase.

2) The propped and unpropped construction were already taken into account in deriving the plastic rotation where strain in bottom flange of beam at midspan was calculated.

Instead, alteration in the elastic rotation is proposed as follows. The initial rotation of the end of the steel beam at construction stage can be written as:

$$\phi_i = (M_{si} + M_{ji})\frac{L}{3EI_s} - \frac{M_{ji}L}{2EI_s}$$

where M_{si} is the sagging moment of steel beam and M_{ji} is the joint moment at the construction stage. The sagging bending moment due to construction load (assumed 10 kN/m) has been checked for several beams with steel connections having stiffnesses as those given in Table 8-6 , using the beam line method, and found to be 1-1.7 times the support moment. Assuming $M_{si} = \frac{4}{3} M_{ji}$, the above equation can be written as:

$$\phi_i = 0.28\,\frac{M_{ji}L}{EI_s}$$

It is further assumed that:

$$M_{ji} = \left(\frac{dead\ \ load}{dead\ \ load + live\ \ load}\right) M_j = 0.3M_j$$

where M_j is the moment resistance of composite joint. Assuming also the ratio of the second moment of area of steel section to uncracked composite section as 0.4, and taking $M_j = k_1 M_{ys}$:

$$\phi_i = 0.21\frac{k_1 M_{ys} L}{3EI} \tag{8.21}$$

To find the elastic rotation at the beam end, the initial rotation of Eqn. (8.21) should be deducted from that of Eqn. (8.12):

$$\phi_e = \frac{M_{ys}L}{3EI}(1-0.71k_1) \tag{8.22}$$

Assuming the shape factor $s = \frac{M_{ps}}{M_{ys}}=1.3$, the elastic rotation will be:

$$\phi_e = 0.33\frac{f_y L}{ED}(1-0.93k) \tag{8.23}$$

The total rotation of the end connection is the sum of the elastic rotation from Eqn. (8.23) and the plastic rotation obtained in Appendix G for unpropped beam:

$$\phi = \frac{f_y L}{ED}[0.33-0.31k+0.55(1+k)^{-1/2}] \tag{8.24}$$

Since the simplifications made in Appendix G are based on Grade 43 steel, Eqn. (8.24) is corrected for Grade 50 steel:

$$\phi = 0.88\frac{f_y L}{ED}[0.33-0.31k+0.55(1+k)^{-1/2}] \tag{8.25}$$

8-5-3-Verification of Proposed Formulae

In order to check the formulae derived in section 8-5-2 against the actual beam behaviour, a number of composite beams comprising composite slabs and semi rigid connections at the beam ends were analysed by the computer program under uniformly distributed load. The selected beams covers the following range:

Steel section	from 203x133x30 UB to 610x229x101 UB.
Steel grade	Grade 43 and Grade 50.
Span	6.0, 9.0 and 12.0 m.
Reinforcement	1% of the area of concrete above decking within the effective breadth, with f_{yr}=460 N/mm^2 and γ_m=1.15.
Concrete	Grade 30 with f_{cu}=30 N/mm^2 and γ_m=1.50.

The effective breadths and subsequent amounts of reinforcement used in the analyses are as given in Table 8-5. The connection behaviour was assumed to be that of the boundary between rigid and semi-rigid connections in EC4 with the connection resistance taken as the fractions of the moment resistance of the beam in hogging bending, as shown in Fig. 8-10(b). The behaviour of steel connection in unpropped analysis was as described in 8-4-3.

In almost all beams with partial strength connections, the first hinge formed at the connection followed by the hinge at the midspan. This was the situation in which the influence of connection flexibility on the required rotation at the beam end would be eliminated. It was described in section 8-5-1 that the required rotation of the beam for a plastic mechanism is independent of the connection stiffness as far as partial strength connections are concerned.

The results of the numerical analysis are compared with the calculated rotation capacities in Tables 8-11 to 8-16. The first three tables belong to propped construction, whilst the other three relate to unpropped construction. Each series consists of 6.0, 9.0 and 12.0 m spans with the span to depth (steel beam) ratio between 15 and 30. In these tables the end rotations have been computed for developing 95% of the plastic resistance of the midspan section.

8-5-3-1-Propped Construction

From Tables 8-11 to 8-13 for propped construction, it is seen that Eqn. (8.18) gives values quite close to the analysis results.

For the 6.0 m span, the calculated values are 5% to 50% greater than the exact values. The equation produces better results for Grade 43 than Grade 50 and also for smaller values of k.

For the 9.0 m span, the difference between calculated and exact values is from -15% to +20% (negative sign means the equation underestimates rotation). Eqn. (8.18) results in closer values of rotation for Grade 50 steel.

For the 12.0 m span, there is a better agreement between calculated and exact values for both grades of steel; the difference being between -12% to +12%.

It is concluded that Eqn. (8.18) results in the closer values of required rotation for the larger spans of 9.0 and 12.0 m which are most likely in construction practice.

8-5-3-2-Unpropped Construction

For unpropped beams, the calculated values using Eqn. (8.25) are compared to the results of exact analysis and those resulted from both the original and simplified equations suggested by Gibbons(1992). It is seen from Tables 8-14 to 8-16 that generally, the Gibbons's original equation (Eqn. (8.19)) underestimates the required rotation particularly for spans of 9.0 m and 12.0 m. His simplified equation (Eqn. (8.20)) gives better results than the original one, the difference to exact rotations being between -54% and 15%.

The calculated values using Eqn. (8.25) are in most cases greater than the exact rotations. In general, better results are obtained compared to those from Gibbons's equations. The difference between the author's results and the exact values is from -10% to +40%.

It is concluded therefore that Eqn. (8.25) proposed by the author gives more accurate and safer values for required rotation. It is safer because underestimation of the end rotation would result in the design of a beam the supports of which are not capable of developing the rotation needed for the assumed plastic behaviour.

8-5-4-Required Rotation for Attainment of Full Sagging Moment Resistance

In the derivation of formulae for the required rotation of connections, it was assumed that if only 95% of sagging moment resistance is reached, this is sufficient in the design of composite beams. In the numerical studies, the required rotation for the attainment of 100% sagging moment resistance has also been found from computer analysis.

The same beams as used in 8-5-3 have been assumed with similar support moments (shown in Fig. 8-9). Sample results are presented in Figs. 8-13 to 8-15 for Grade 50 steel and spans of 6.0, 9.0 and 12.0 m. These figures relate to propped construction.

As mentioned earlier, the grade of steel has a modest influence on the amount of required rotation. This is because the shape of bending moment diagram and the gradient of moment is similar for both grades. Furthermore, the connection strength in these analyses has been assumed proportional to the moment resistance of the beam in hogging (or sagging). Thus, unique charts may conservatively be presented for the both grades of steel by taking the greater values of rotation which occurred in the beams of Grade 50 steel.

In order to produce a chart suitable for design purposes, the computed results of 9.0 m span beam are taken. An empirical factor is introduced in terms of the beam length to take account of the various spans, as described below. The chart of Fig. 8-16 has been obtained by modifying the plots of Fig. 8-14 for span to depth ratios from 15 to 30. Having assumed a support moment with respect to the plastic moment resistance of the composite beam in sagging bending, associated with a known span to depth ratio, a value for rotation capacity can be found, to assist in designing the end connections. The rotation found from this chart shoud be multiplied by factor s_m to be corrected for the beam length:

$$s_m = \left(\frac{L}{9} \right)^{0.4}$$

where L is the beam span in meters. This factor covers composite beams with 1% reinforcement having a span of between 6.0 to 12.0 m under uniformly distributed load. The concrete grade would not be influential since in the author's investigations the depth of concrete in compression has always been found much smaller than the depth of concrete in the slab.

Sample checks for 6.0 and 9.0 m spans are summarized in Table 8-17. As seen from the table, the chart generally gives conservative values for the required rotation. A similar chart may be produced for unpropped beams.

8-6-Conclusions

In this chapter, methods of analysis of composite beams were considered. Redistribution of moments in elastic analysis and the required rotation capacity in plastic analysis were discussed. A computer program was introduced and used to study these subjects. The longitudinal reinforcement was proposed to have a total area of 1% with respect to the area of concrete above decking within the effective breadth (as given by EC4 and BS5950:Pt 3.1) to provide optimum rotation capacity. The following conclusions can be drawn from the numerical analyses of composite beams with 1% reinforcement:

1) Adopting a greater steel section for the composite beam of a specified span will reduce the required rotation capacity of the end connections for a full plastic mechanism.
2) Generally, greater spans require more rotation capacity for the end connections in order to develop a full plastic mechanism.
3) More redistribution of moments in composite beams necessitates more rotation capacity for the end connections.
4) The maximum percentage of redistribution of moments given as 40% in EC4 and BS5950:Pt 3.1 is an appropriate value for both propped and unpropped composite beams. Although more redistribution of moments may be possible in conservatively designed beams, values greater than 40% are not advisable.

5) Greater percentages of redistribution than those given by EC4 and BS5950:Pt 3.1 may be assumed for non-Plastic composite sections if further justification is provided. This is based on the satisfactory behaviour of Class 3 composite sections subjected to negative moment in the author's tests.

6) It is possible to reduce the rotation requirements of connections by not utilizing the full sagging moment resistance of the composite beam.

7) The flexibility of partial strength connections does not influence the rotation needed for plastic mechanism, rather its resistance determines the required rotation. Very flexible connections may cause the first hinge to form at midspan.

8) The required rotation of the end connections can be found from the elastic and plastic rotations of composite beam. The expressions proposed by the author give generally safe and sufficiently accurate estimations of the required rotation.

9) Design charts can be provided for determination of the required rotation capacity in design of composite connections of semi-continuous frames.

Table 8-1, Maximum percentage of redistribution of support moment in BS5950:Pt 3.1 and EC4:Part 1

CODE	METHOD OF ANALYSIS	Class 1 Plastic		Class 2	Class 3 Semi-Compact	Class 4
		Non-reinforced	Generally	Compact		Slender
BS5950 Pt 3.1	Uncracked	50	40	30	20	10
	Cracked	40	30	20	10	0
EC4 Part 1	Uncracked	40	40	30	20	10
	Cracked	25	25	15	10	0

Table 8-2, Results of the pilot study on redistribution of moments (Test 1)

GRADE OF STEEL	f_{ys} N/mm^2	M_{cc} kNm	M_{ph} kNm	M_{ps} kNm	UDL kN/m	REDISTRIBUTION PERCENTAGE %	ϕ $mrad$
43	275	219	292	329	54	40	22.9
50	355	219	333	419	63	49	32.3
55	450	219	434	508	72	55	35.9

Table 8-3, Results of the pilot study on redistribution of moments (Test 10)

GRADE OF STEEL	f_{ys} N/mm^2	M_{cc} kNm	M_{ph} kNm	M_{ps} kNm	UDL kN/m	REDISTRIBUTION PERCENTAGE %	ϕ $mrad$
43	275	328	467	554	53	44	24.2
50	355	328	570	717	63	53	36.9
55	450	328	639	872	73	59	37.9

Table 8-4, Results of analysis of beams with variable reinforcement

REINFORCEMENT PERCENTAGE %	M_{cc} kNm	M_{ph} kNm	M_{ps} kNm	REQUIRED ROTATION mrad
0.5 (Test 3)	179	251	329	21.7
1.0 (Test 1)	262	292	329	14.0
1.5 (Test 7)	302	326	329	10.0

Table 8-5, The effective breadths and amounts of reinforcement used in verification of proposed formulae

SPAN m	EFFECTIVE BREADTH mm	AMOUNT OF REINFORCEMENT mm^2
6.0	750	555
9.0	1125	832.5
12.0	1500	1110

Note-The amounts of reinforcement have been calculated as 1% of the area of concrete above decking found from multiplying (120-46)=74 mm by the effective breadth.

Table 8-6, Stiffnesses of steelwork connections used in unpropped analysis (values are in kNm/mrad)

STEEL SECTION	CONNECTION STIFFNESS	
	GRADE 43	GRADE 50
203x133 UB 30	10.2	11.6
254x146 UB 37	12.7	14.4
305x165 UB 40	15.3	17.4
356x171 UB 45	17.9	20.3
406x140 UB 46	20.4	23.2
457x152 UB 52	22.9	26.0
533x210 UB 82	26.7	30.4
610x229 UB 101	30.6	34.8

Table 8-7, Results of 40% moment redistribution

SPAN	STEEL SECTION	GRADE 43							GRADE 50						
					Propped		Unpropped					Propped		Unpropped	
m		M_{ph} kNm	M_{ps} kNm	M_{pc} kNm	w kN/m	ϕ mrad	w kN/m	ϕ mrad	M_{ph} kNm	M_{ps} kNm	M_{pc} kNm	w kN/m	ϕ mrad	w kN/m	ϕ mrad
6.0	203x133x30	145	186	125	69	19.2	58	21.7	175	220	147	82	19.1	71	22.6
	254x146x37	204	248	166	92	17.3	82	18.7	224	295	198	110	17.0	99	18.7
	305x165x40	253	297	198	110	16.7	100	16.3	290	357	238	132	15.4	122	15.4
	356x171x45	295	356	237	132	14.1	123	16.2	394	430	287	159	14.1	150	14.6
9.0	305x165x40	273	321	215	53	24.9	43	29.9	345	397	266	65	27.8	55	30.7
	356x171x45	329	392	262	65	25.8	54	28.7	401	475	318	78	23.8	68	25.4
	406x140x46	361	445	298	73	24.7	63	25.7	457	542	363	89	22.7	79	25.9
	457x152x52	448	531	356	88	20.8	79	28.0	570	649	435	107	20.0	97	20.3
12.0	406x140x46	399	452	303	42	27.2	32	36.4	499	569	381	53	29.1	42	37.8
	457x152x52	474	548	367	51	26.5	41	33.9	598	688	461	64	30.3	53	32.4
	533x210x82	778	927	621	86	28.1	76	27.7	1037	1123	753	104	23.3	94	24.9
	610x229x101	1093	1230	823	114	23.4	104	25.0	1424	1502	1006	139	19.3	129	21.5

Table 8-8, Results of 50% moment redistribution

SPAN	STEEL SECTION	GRADE 43							GRADE 50						
		M_{ph}	M_{ps}	M_{pc}	Propped		Unpropped		M_{ph}	M_{ps}	M_{pc}	Propped		Unpropped	
					w	ϕ	w	ϕ				w	ϕ	w	ϕ
m		*kNm*	*kNm*	*kNm*	*kN/m*	*mrad*	*kN/m*	*mrad*	*kNm*	*kNm*	*kNm*	*kN/m*	*mrad*	*kN/m*	*mrad*
6.0	203x133x30	145	186	93	62	24.1	51	29.8	175	220	147	110	23.4	62	27.3
	254x146x37	204	248	124	83	19.0	72	23.5	224	295	148	98	20.2	88	22.1
	305x165x40	253	297	148	99	18.9	88	20.3	290	357	178	119	18.2	109	18.0
	356x171x45	295	356	178	119	16.2	109	18.3	394	430	215	143	16.2	133	15.7
9.0	305x165x40	273	321	161	48	28.3	37	34.1	345	397	198	59	31.4	48	35.4
	356x171x45	329	392	196	58	28.2	47	30.1	401	475	238	70	27.2	60	31.5
	406x140x46	361	445	223	66	28.1	56	29.2	457	542	271	80	24.9	70	27.3
	457x152x52	448	531	265	79	22.5	70	28.9	570	649	325	96	23.0	86	23.4
12.0	406x140x46	399	452	226	38	30.7	27	41.3	499	569	285	47	34.9	37	42.9
	457x152x52	474	548	274	46	28.8	36	38.3	598	688	344	57	34.4	46	37.0
	533x210x82	778	927	463	77	31.2	67	33.9	1037	1123	562	94	26.5	84	26.1
	610x229x101	1093	1230	615	102	26.1	93	27.7	1424	1502	751	125	24.0	115	22.5

Table 8-9, Results of the initial studies on required rotation

SPAN	6.0 m								7.5 m							
SECTION	178x102x19		203x133x30		254x146x37		305x165x40		203x133x30		254x146x37		305x165x40		356x171x45	
$\bar{m}$	*w* *kN/m*	ϕ *mrad*	*w* *kN/m*	ϕ *mrad*	*w* *kN/m*	ϕ *mrad*	*w* *kN/m*	ϕ *mrad*	*w* *kN/m*	ϕ *mrad*	*w* *kN/m*	ϕ *mrad*	*w* *kN/m*	ϕ *mrad*	*w* *kN/m*	ϕ *mrad*
0.8	42	17.9	67	21.4	92	16.8	111	16.4	44	21.7	63	24.5	74	25.1	91	19.6
0.7	40	19.6	64	23.0	88	18.3	105	17.7	42	23.7	59	26.7	70	26.6	87	21.2
0.6	38	21.0	61	24.2	83	19.5	100	18.7	40	25.4	56	28.2	67	28.0	82	22.6
0.5	36	22.5	57	26.1	78	21.0	94	19.5	37	26.6	53	29.2	63	29.3	77	23.9
0.4	34	24.0	54	27.4	74	22.7	88	20.9	35	28.3	50	31.4	60	30.8	72	24.9

SPAN	9.0 m								11.5 m							
SECTION	305x165x40		356x171x45		406x140x46		457x152x52		305x165x40		356x171x45		406x140x46		457x152x52	
$\bar{m}$	*w* *kN/m*	ϕ *mrad*	*w* *kN/m*	ϕ *mrad*	*w* *kN/m*	ϕ *mrad*	*w* *kN/m*	ϕ *mrad*	*w* *kN/m*	ϕ *mrad*	*w* *kN/m*	ϕ *mrad*	*w* *kN/m*	ϕ *mrad*	*w* *kN/m*	ϕ *mrad*
0.8	53	24.5	65	25.1	73	24.4	88	21.0	34	26.5	41	25.2	46	25.3	56	24.9
0.7	51	26.4	61	26.6	69	24.6	83	22.4	32	29.4	39	28.1	44	27.2	53	26.2
0.6	48	28.0	58	28.9	65	27.4	79	23.7	31	31.7	37	29.3	42	28.9	50	28.2
0.5	45	29.0	55	30.5	62	29.3	75	24.9	29	33.9	34	31.7	39	30.6	47	29.8
0.4	43	31.0	52	30.9	58	30.0	70	25.7	27	35.6	32	33.5	37	32.3	44	30.5

Table 8-10, Rotations resulted from analysis of composite beams with variable connection stiffness and resistance

β	$\bar{m}$	305x165 UB 40 Span=9.0 m		457x152 UB 52 Span=11.5 m	
		w *kN/m*	ϕ *mrad*	*w* *kN/m*	ϕ *mrad*
1.0	0.8	53	24.5	56	24.9
	0.7	51	26.4	53	26.2
	0.6	48	28.0	50	28.2
	0.5	45	29.0	47	29.8
	0.4	43	31.0	44	30.5
1.5	0.8	53	24.5	56	24.9
	0.7	51	26.3	53	26.3
	0.6	48	27.4	50	28.2
	0.5	45	29.3	47	29.8
	0.4	43	30.5	44	31.1
2.0	0.8	53	24.5	56	24.9
	0.7	51	26.3	53	26.4
	0.6	48	27.7	50	28.2
	0.5	45	28.9	47	29.8
	0.4	43	31.1	44	30.6
4.0	0.8	*	*	*	*
	0.7	*	*	53	26.6
	0.6	48	27.7	50	28.2
	0.5	45	29.0	47	29.8
	0.4	43	30.5	44	30.3

* First hinge formed at midspan.

Table 8-11, Calculated required rotations compared to analysis results of propped beams with 6.0 m span

STEEL SECTION	$\frac{L}{D}$	GRADE 43 M_{pc} kNm	GRADE 43 k	GRADE 43 ϕ_{ex} mrad	GRADE 43 (8.18) mrad	GRADE 50 M_{pc} kNm	GRADE 50 k	GRADE 50 ϕ_{ex} mrad	GRADE 50 (8.18) mrad
203x133x30	29.0	116	0.62	15.3	19.4	140	0.64	14.4	21.8
		101	0.54	17.0	20.3	123	0.56	16.4	22.9
		87	0.47	18.3	21.2	105	0.48	17.9	24.0
		72	0.39	19.5	22.2	88	0.40	19.3	25.1
		58	0.31	20.7	23.2	70	0.32	20.7	26.3
		43	0.23	22.0	24.3	53	0.24	22.0	27.5
254x146x37	23.4	163	0.66	12.0	15.3	179	0.61	13.2	17.9
		143	0.58	13.5	16.1	157	0.53	14.4	18.8
		122	0.49	14.6	16.9	135	0.46	15.6	19.6
		102	0.41	15.7	17.7	112	0.38	16.7	20.5
		82	0.33	16.7	18.6	90	0.30	17.8	21.4
		61	0.25	17.7	19.4	67	0.23	18.8	22.3
305x165x40	19.7	202	0.68	10.5	12.7	232	0.65	11.0	14.7
		177	0.60	11.6	13.4	203	0.57	12.1	15.5
		152	0.51	12.6	14.1	174	0.49	13.1	16.2
		126	0.43	13.5	14.8	145	0.41	14.1	17.0
		101	0.34	14.4	15.5	116	0.33	15.1	17.8
		76	0.26	15.3	16.3	87	0.24	16.0	18.6
356x171x45	17.0	236	0.66	9.4	11.1	315	0.73	8.9	12.1
		207	0.58	10.3	11.7	276	0.64	10.3	12.8
		177	0.50	11.1	12.3	237	0.55	11.3	13.5
		148	0.41	11.9	12.9	197	0.46	12.4	14.2
		118	0.33	12.7	13.5	158	0.37	13.4	15.0
		89	0.25	13.4	14.1	118	0.27	14.3	15.8

Table 8-12, Calculated required rotations compared to analysis results of propped beams with 9.0 m span

STEEL SECTION	$\frac{L}{D}$	GRADE 43				GRADE 50			
		M_{pc} kNm	k	ϕ_{ex} mrad	(8.18) mrad	M_{pc} kNm	k	ϕ_{ex} mrad	(8.18) mrad
305x165x40	29.6	219	0.68	16.6	19.1	276	0.69	17.5	21.5
		191	0.60	18.4	20.1	241	0.61	19.8	22.7
		164	0.51	19.9	21.1	207	0.52	21.5	23.9
		137	0.43	21.3	22.2	172	0.43	23.2	25.1
		109	0.34	22.7	23.3	138	0.35	24.9	26.4
		82	0.26	24.0	24.5	103	0.26	26.5	27.7
356x171x45	25.6	263	0.67	16.3	16.6	321	0.68	15.7	18.8
		230	0.59	17.9	17.4	281	0.59	17.4	19.8
		197	0.50	19.2	18.3	241	0.51	18.9	20.8
		164	0.42	20.4	19.2	201	0.42	20.3	21.8
		131	0.34	21.7	20.2	160	0.34	21.8	22.9
		99	0.25	23.0	21.2	120	0.25	23.1	24.0
406x140x46	22.4	289	0.65	15.6	14.7	366	0.68	14.7	16.4
		253	0.57	16.8	15.4	320	0.59	16.2	17.3
		217	0.49	18.0	16.2	274	0.51	17.6	18.2
		181	0.41	19.1	17.0	229	0.42	18.9	19.1
		145	0.32	20.2	17.8	183	0.34	20.1	20.0
		108	0.24	21.4	18.6	137	0.25	21.3	21.0
457x152x52	20.0	358	0.67	12.8	12.9	456	0.70	12.8	14.4
		313	0.59	14.0	13.6	399	0.61	14.3	15.2
		269	0.51	15.1	14.3	342	0.53	15.6	16.1
		224	0.42	16.1	15.0	285	0.44	16.8	16.9
		179	0.34	17.2	15.8	228	0.35	18.0	17.8
		134	0.25	18.2	16.5	171	0.26	19.2	18.7

Table 8-13, Calculated required rotations compared to analysis results of propped beams with 12.0 m span

STEEL SECTION	$\frac{L}{D}$	GRADE 43				GRADE 50			
		M_{pc} kNm	k	ϕ_{ex} mrad	(8.18) mrad	M_{pc} kNm	k	ϕ_{ex} mrad	(8.18) mrad
406x140x46	29.8	319	0.71	16.9	18.9	399	0.70	19.5	21.6
		279	0.62	18.8	20.0	349	0.61	21.9	22.7
		239	0.53	20.4	21.1	299	0.53	23.8	24.0
		199	0.44	21.8	22.2	249	0.44	25.6	25.2
		159	0.35	23.3	23.3	200	0.35	27.4	26.5
		120	0.26	24.7	24.5	150	0.26	29.1	27.9
457x152x52	26.7	379	0.69	16.3	17.1	479	0.70	18.4	19.3
		332	0.61	17.9	18.0	419	0.61	20.4	20.4
		284	0.52	19.3	18.9	359	0.52	22.2	21.5
		237	0.43	20.6	19.9	299	0.43	23.8	22.6
		189	0.35	22.0	20.9	239	0.35	25.5	23.8
		142	0.26	23.3	22.0	179	0.26	27.1	25.0
533x210x82	22.7	622	0.67	15.8	14.7	830	0.74	14.1	16.0
		544	0.59	17.1	15.5	726	0.65	16.0	17.0
		467	0.50	18.3	16.3	622	0.55	17.5	18.0
		389	0.42	19.5	17.1	519	0.46	19.0	19.0
		311	0.34	20.7	17.9	415	0.37	20.4	20.0
		233	0.25	21.9	18.8	311	0.28	21.8	21.1
610x229x101	19.9	913	0.74	12.1	12.4	1139	0.76	12.6	13.9
		799	0.65	13.6	13.1	997	0.66	14.4	14.7
		684	0.56	14.8	13.8	855	0.57	15.8	15.6
		570	0.46	15.9	14.6	712	0.47	17.2	16.5
		456	0.37	17.1	15.4	570	0.38	18.5	17.4
		342	0.28	18.1	16.2	427	0.28	19.8	18.4

Table 8-14, Calculated required rotations compared to analysis results of unpropped beams with 6.0 m span

STEEL SECTION	$\frac{L}{D}$	GRADE 43						GRADE 50					
		M_{pc} kNm	k	ϕ_{ex} mrad	Gibbons (8.19) mrad	Gibbons (8.20) mrad	Author (8.25) mrad	M_{pc} kNm	k	ϕ_{ex} mrad	Gibbons (8.19) mrad	Gibbons (8.20) mrad	Author (8.25) mrad
203x133x30	29.0	116	0.62	20.1	17.2	19.7	22.2	140	0.64	17.6	21.9	25.2	24.7
		101	0.54	22.1	18.0	20.3	23.6	123	0.56	19.7	22.9	26.0	26.3
		87	0.47	23.7	18.7	20.9	25.0	105	0.48	21.5	23.9	26.8	27.9
		72	0.39	25.3	19.5	21.5	26.4	88	0.40	23.2	24.9	27.6	29.6
		58	0.31	26.8	20.3	22.1	27.8	70	0.32	24.9	26.0	28.4	31.3
		43	0.23	28.5	21.2	22.7	29.3	53	0.24	26.7	27.2	29.2	33.1
254x146x37	23.4	163	0.66	14.5	13.7	15.7	17.5	179	0.61	14.4	17.9	20.5	20.3
		143	0.58	15.9	14.3	16.2	18.7	157	0.53	15.8	18.7	21.1	21.5
		152	0.49	17.2	15.0	16.8	19.8	135	0.46	17.1	19.5	21.8	22.8
		102	0.41	18.4	15.6	17.3	21.0	112	0.38	18.4	20.3	22.4	24.1
		82	0.33	19.7	16.3	17.8	22.3	90	0.30	19.7	21.1	23.0	25.5
		61	0.25	20.9	17.0	18.3	23.5	67	0.23	21.0	22.0	23.7	26.9
305x165x40	19.7	202	0.68	11.5	11.3	13.1	14.3	232	0.65	11.7	14.8	17.1	16.7
		177	0.60	12.7	11.9	13.5	15.3	203	0.57	12.8	15.5	17.6	17.8
		152	0.51	13.7	12.4	14.0	16.4	174	0.49	14.0	16.2	18.2	18.9
		126	0.43	14.7	13.0	14.4	17.4	145	0.41	15.0	16.9	18.7	20.0
		101	0.34	15.8	13.6	14.9	18.5	116	0.33	16.1	17.7	19.3	21.2
		76	0.26	16.8	14.2	15.3	19.6	87	0.24	17.2	18.5	19.9	22.5
356x171x45	17.0	236	0.66	10.6	10.0	11.5	12.8	315	0.73	8.9	12.2	14.2	13.4
		207	0.58	11.4	10.4	11.8	13.6	276	0.64	10.5	12.8	14.7	14.4
		177	0.50	12.3	10.9	12.2	14.4	237	0.55	11.6	13.5	15.3	15.5
		148	0.41	13.2	11.4	12.6	15.3	197	0.46	12.7	14.2	15.8	16.6
		118	0.33	14.0	11.9	12.9	16.2	158	0.37	13.8	14.9	16.4	17.8
		89	0.25	14.8	12.4	13.3	17.1	118	0.27	14.9	15.6	16.9	18.9

Table 8-15, Calculated required rotations compared to analysis results of unpropped beams with 9.0 m span

STEEL SECTION	$\frac{L}{D}$	GRADE 43						GRADE 50					
		M_{pc} kNm	k	ϕ_{ex} mrad	Gibbons (8.19) mrad	Gibbons (8.20) mrad	Author (8.25) mrad	M_{pc} kNm	k	ϕ_{ex} mrad	Gibbons (8.19) mrad	Gibbons (8.20) mrad	Author (8.25) mrad
		219	0.68	22.2	16.9	19.6	21.4	276	0.69	21.9	21.8	25.2	24.2
		191	0.60	24.5	17.7	20.2	22.9	241	0.61	24.4	22.8	26.1	25.9
305x165x40	29.6	164	0.51	26.4	18.6	20.9	24.5	207	0.52	26.5	23.9	27.0	27.7
		137	0.43	28.3	19.4	21.6	26.1	172	0.43	28.6	25.1	27.9	29.5
		109	0.34	30.1	20.4	22.3	27.7	138	0.35	30.7	26.2	28.7	31.4
		82	0.26	32.1	21.3	23.0	29.4	103	0.26	32.8	27.5	29.6	33.4
		263	0.67	18.4	14.5	16.8	18.3	321	0.68	18.2	18.9	21.8	21.0
		230	0.59	20.4	15.2	17.4	19.6	281	0.59	20.2	19.8	22.6	22.5
356x171x45	25.6	197	0.50	22.0	15.9	18.0	21.0	241	0.51	21.9	20.7	23.3	24.0
		164	0.42	23.6	16.7	18.6	22.4	201	0.42	23.6	21.7	24.1	25.6
		131	0.34	25.3	17.5	19.2	23.8	160	0.34	25.3	22.7	24.8	27.2
		99	0.25	26.9	18.3	19.8	25.3	120	0.25	27.0	23.7	25.6	28.8
		289	0.65	17.9	12.8	14.8	16.3	366	0.68	17.0	16.7	19.2	18.7
		253	0.57	19.5	13.4	15.3	17.4	320	0.59	18.6	17.5	19.9	20.0
406x140x46	22.4	217	0.49	20.9	14.1	15.8	18.6	274	0.51	20.1	18.3	20.5	21.3
		181	0.41	22.3	14.7	16.3	19.8	229	0.42	21.5	19.1	21.2	22.6
		145	0.32	23.8	15.4	16.9	21.0	183	0.34	23.1	20.0	21.8	24.0
		108	0.24	25.2	16.1	17.4	22.3	137	0.25	24.5	20.9	22.5	25.4
		358	0.67	17.2	11.5	13.3	14.6	456	0.70	14.0	14.7	17.0	16.3
		313	0.59	18.7	12.0	13.7	15.6	399	0.61	15.6	15.4	17.6	17.5
457x152x52	20.0	269	0.51	20.0	12.6	14.2	16.6	342	0.53	17.1	16.1	18.2	18.7
		224	0.42	21.3	13.2	14.6	17.7	285	0.44	18.5	16.9	18.8	19.9
		179	0.34	22.6	13.8	15.1	18.8	228	0.35	19.9	17.7	19.4	21.2
		134	0.25	23.9	14.4	15.5	19.9	171	0.26	21.2	18.5	20.0	22.5

Table 8-16, Calculated required rotations compared to analysis results of unpropped beams with 12.0 m span

STEEL SECTION	$\frac{L}{D}$	GRADE 43						GRADE 50					
		M_{pc} kNm	k	ϕ_{ex} mrad	Gibbons (8.19) mrad	Gibbons (8.20) mrad	Author (8.25) mrad	M_{pc} kNm	k	ϕ_{ex} mrad	Gibbons (8.19) mrad	Gibbons (8.20) mrad	Author (8.25) mrad
406x140x46	29.8	319	0.71	26.2	19.2	21.5	25.6	399	0.70	24.6	21.7	25.2	24.0
		279	0.62	27.5	19.8	21.9	26.7	349	0.61	27.5	22.8	26.1	25.8
		239	0.53	28.8	20.4	22.4	27.8	299	0.53	29.9	23.9	27.0	27.6
		199	0.44	30.1	21.1	22.9	28.9	249	0.44	32.2	25.1	27.9	29.5
		159	0.35	31.5	21.7	23.3	30.1	200	0.35	34.6	26.3	28.9	31.4
		120	o.26	32.8	22.3	23.7	31.4	150	0.26	37.0	27.6	29.8	33.4
457x152x52	26.7	379	0.69	21.3	15.0	17.4	18.9	479	0.70	22.4	19.6	22.7	21.8
		332	0.61	23.6	15.8	18.1	20.3	419	0.61	24.7	20.6	23.5	23.4
		284	0.52	25.5	16.6	18.7	21.7	359	0.52	26.8	21.5	24.3	25.0
		237	0.43	27.4	17.4	19.4	23.2	299	0.43	28.8	22.6	25.1	26.6
		189	0.35	29.3	18.2	20.0	24.7	239	0.35	30.9	23.6	25.9	28.3
		142	0.26	31.3	19.1	20.6	26.3	179	0.26	32.9	24.7	26.7	30.0
533x210x82	22.7	622	0.67	18.0	13.1	15.1	16.7	830	0.74	14.6	16.3	18.9	17.9
		544	0.59	19.5	13.7	15.6	17.8	726	0.65	16.7	17.1	19.7	19.3
		467	0.50	20.8	14.4	16.1	19.0	622	0.55	18.3	18.0	20.4	20.7
		389	0.42	22.2	15.0	16.6	20.2	519	0.46	20.0	18.9	21.1	22.2
		311	0.34	23.6	15.7	17.2	21.4	415	0.37	21.6	19.9	21.9	23.7
		233	0.25	25.0	16.4	17.7	22.7	311	0.28	23.2	20.9	22.6	25.3
610x229x101	19.9	913	0.74	13.4	11.2	13.0	14.1	1139	0.76	11.8	14.0	16.4	15.3
		799	0.65	14.7	11.8	13.5	15.1	997	0.66	14.2	14.8	17.1	16.5
		684	0.56	16.0	12.4	14.0	16.2	855	0.57	15.7	15.6	17.7	17.8
		570	0.46	17.1	13.0	14.4	17.3	712	0.47	17.1	16.4	18.4	19.2
		456	0.37	18.4	13.6	14.9	18.5	570	0.38	18.6	17.3	19.1	20.6
		342	0.28	19.6	14.2	15.4	19.6	427	0.28	20.0	18.2	19.7	22.0

Table 8-17, Sample check for various beam spans using the chart of Fig. 8-16 for Grade 50 steel and $k=0.5$

SPAN m	s_m	$\frac{L}{D}$	ϕ_{ex} $mrad$	ϕ_{chart} $mrad$
6.0	0.85	16.9	16.0	17.9
		19.7	18.5	19.5
		23.6	19.7	21.8
		29.6	24.0	26.7
12.0	1.12	19.9	24.2	26.1
		22.7	26.7	29.5
		26.7	33.5	33.5
		29.8	36.1	36.0

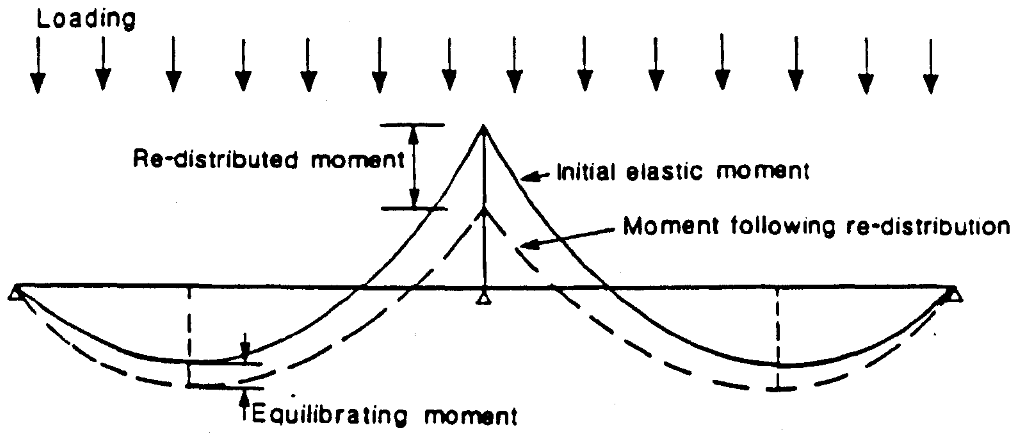


Fig. 8-1, Elastic redistribution of moments

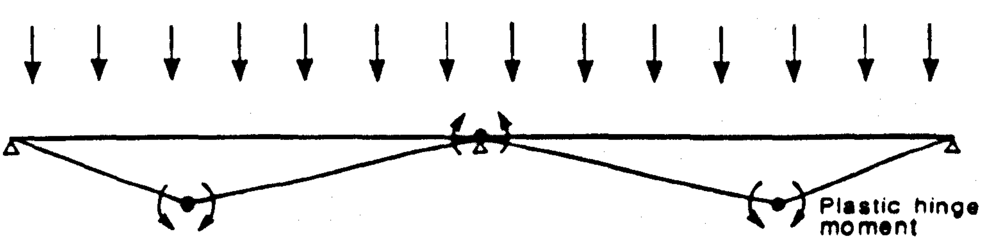
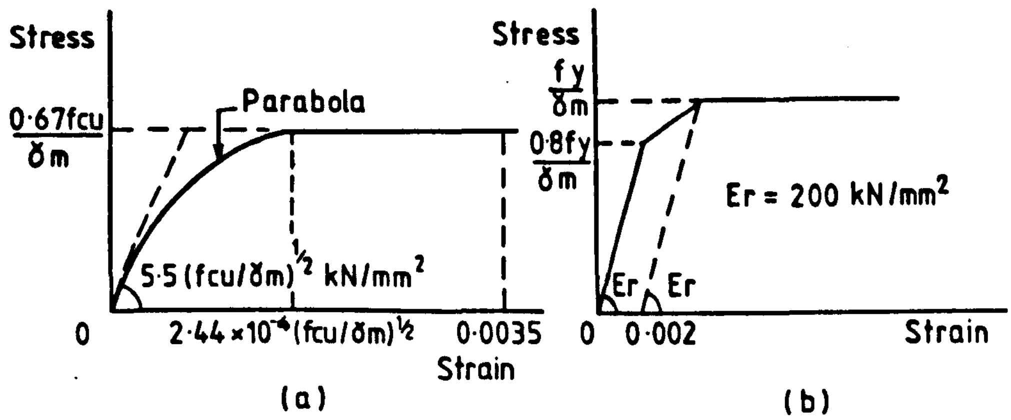
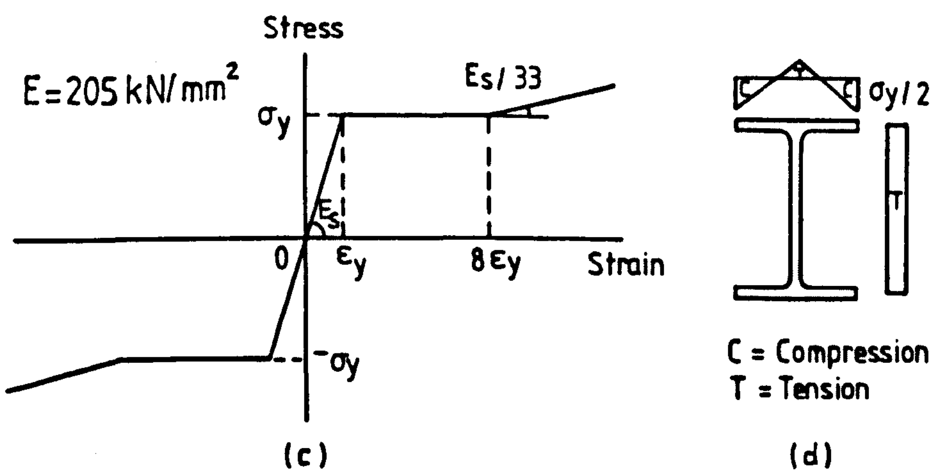


Fig. 8-2, Plastic hinge mechanism

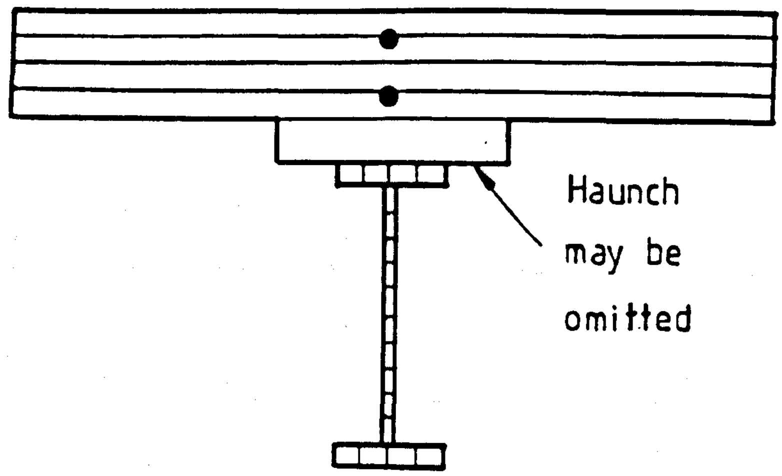


Stress–strain curves; (a) concrete; (b) reinforcement

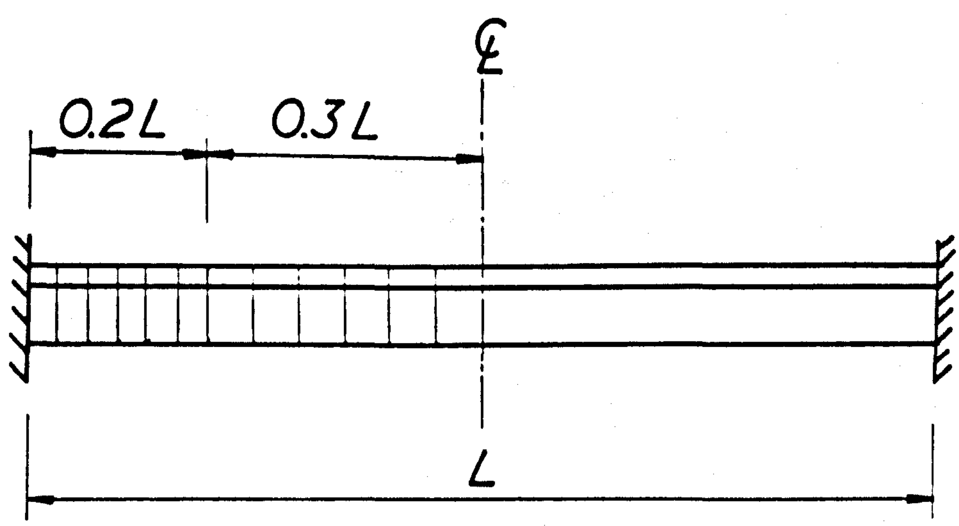


Structural steel material properties (c) stress–strain curve; (d) residual stress pattern

Fig. 8-3, Material properties used in numerical analysis



(a)



(b)

Fig. 8-4, (a) Slices of composite section, (b) Hogging and sagging elements

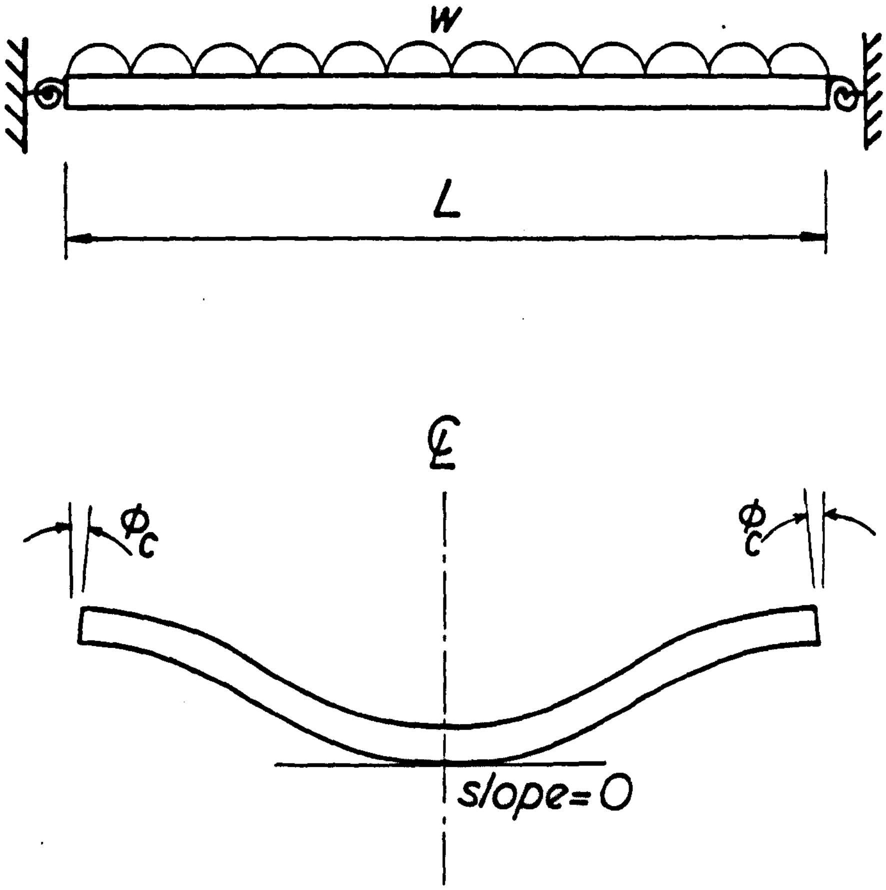


Fig. 8-5, Beam with semi-rigid connections and its deformed shape

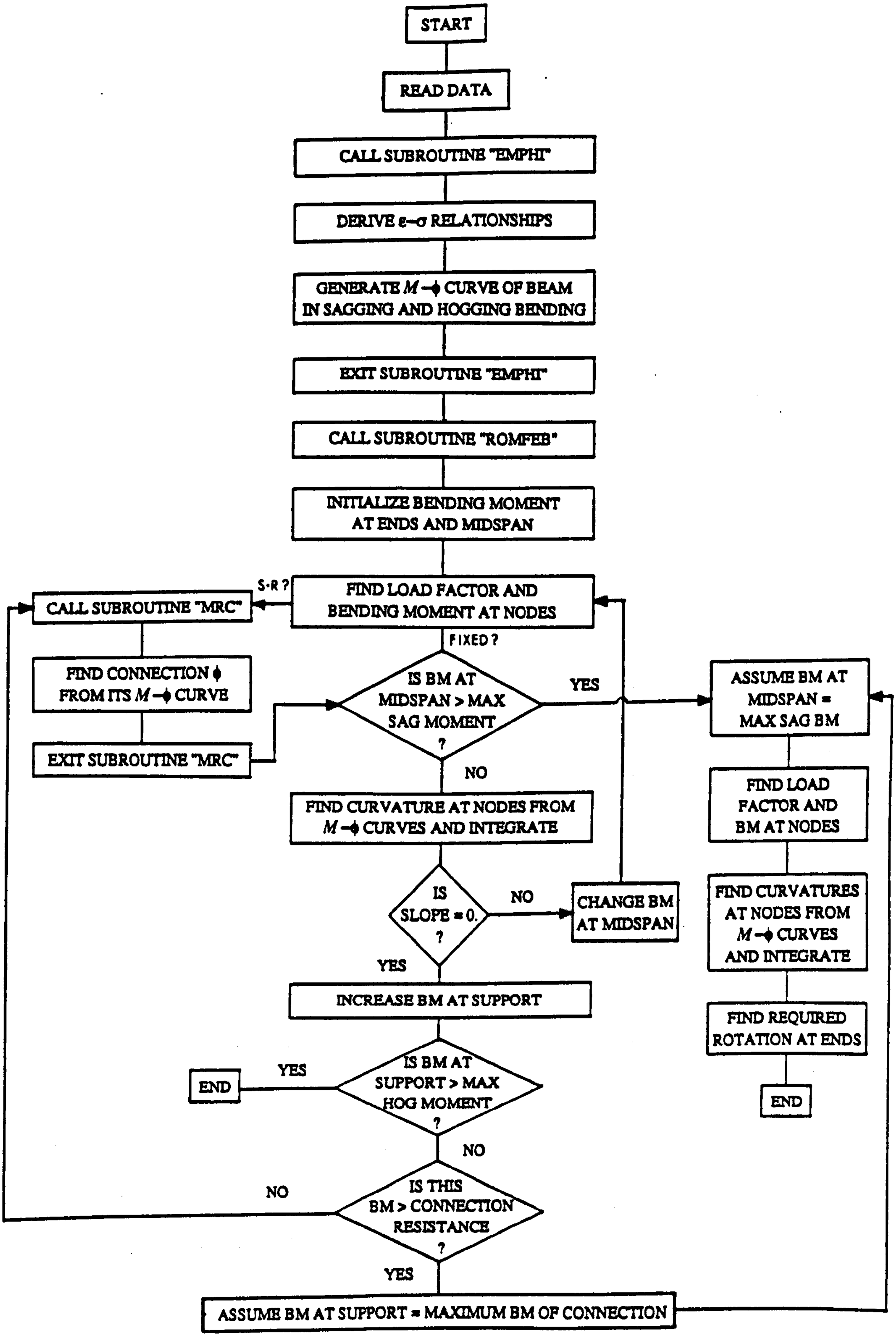


Fig. 8-6, Flowchart of computer program "SRCB"

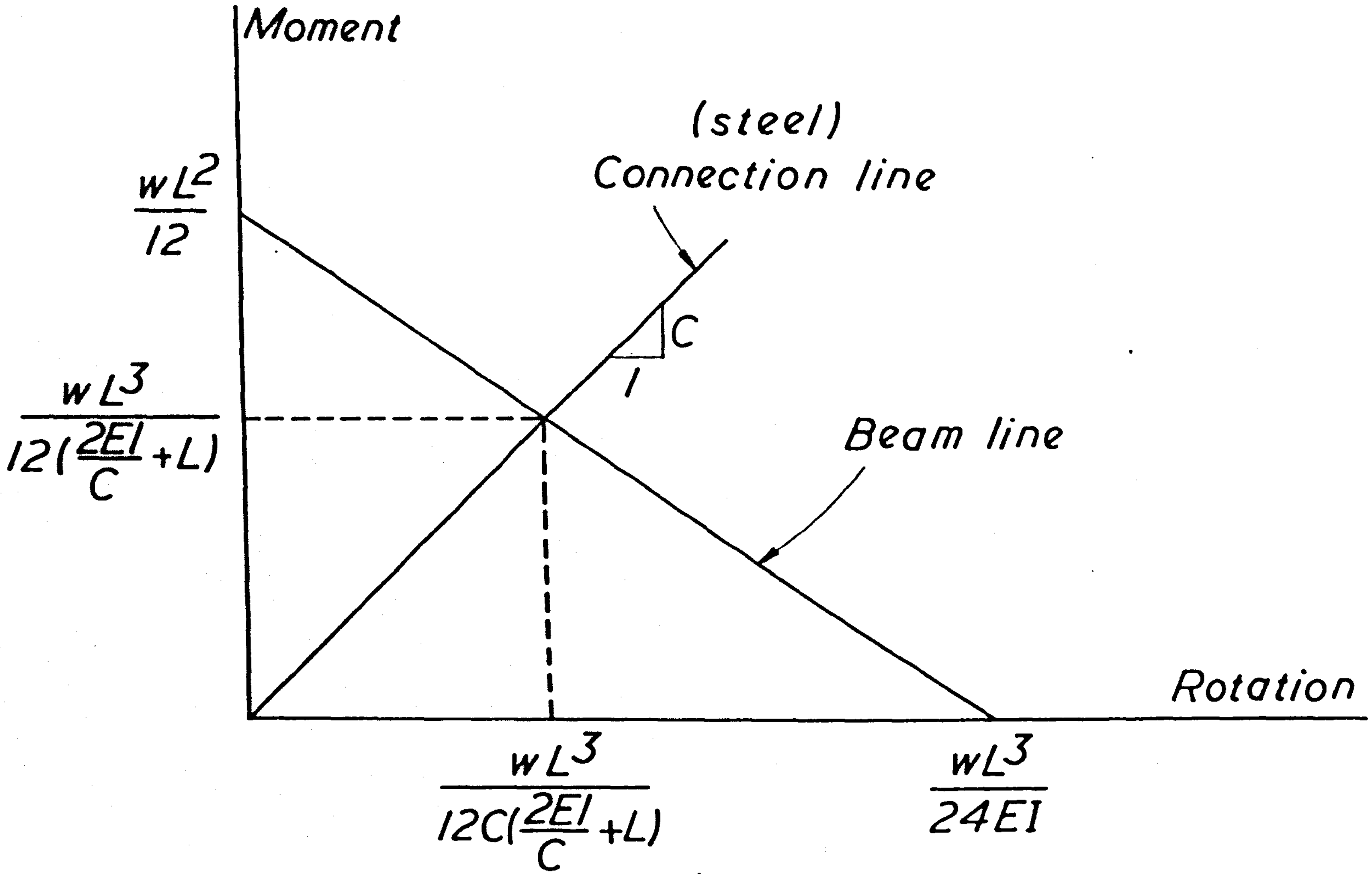


Fig. 8- 7, Beam line method used for initial rotation of steel connection in unpropped construction

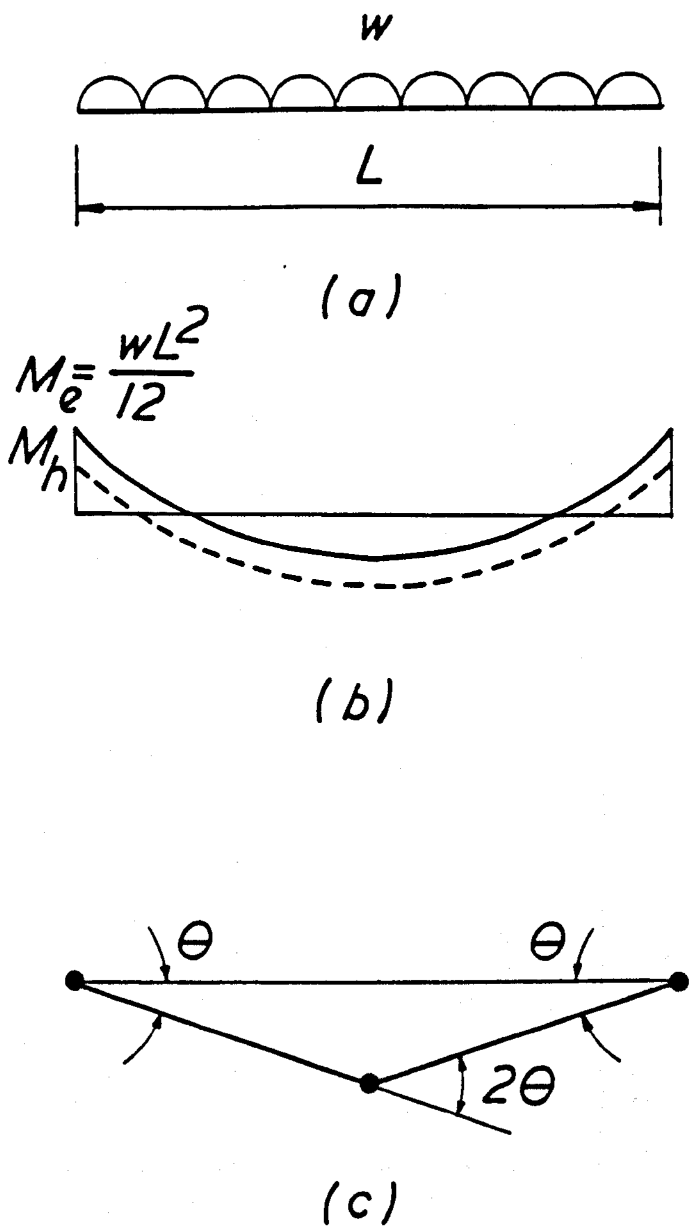


Fig. 8-8, (a) Composite beam with load w and span L
(b) Elastic redistribution of moments
(c) Plastic hinge mechanism

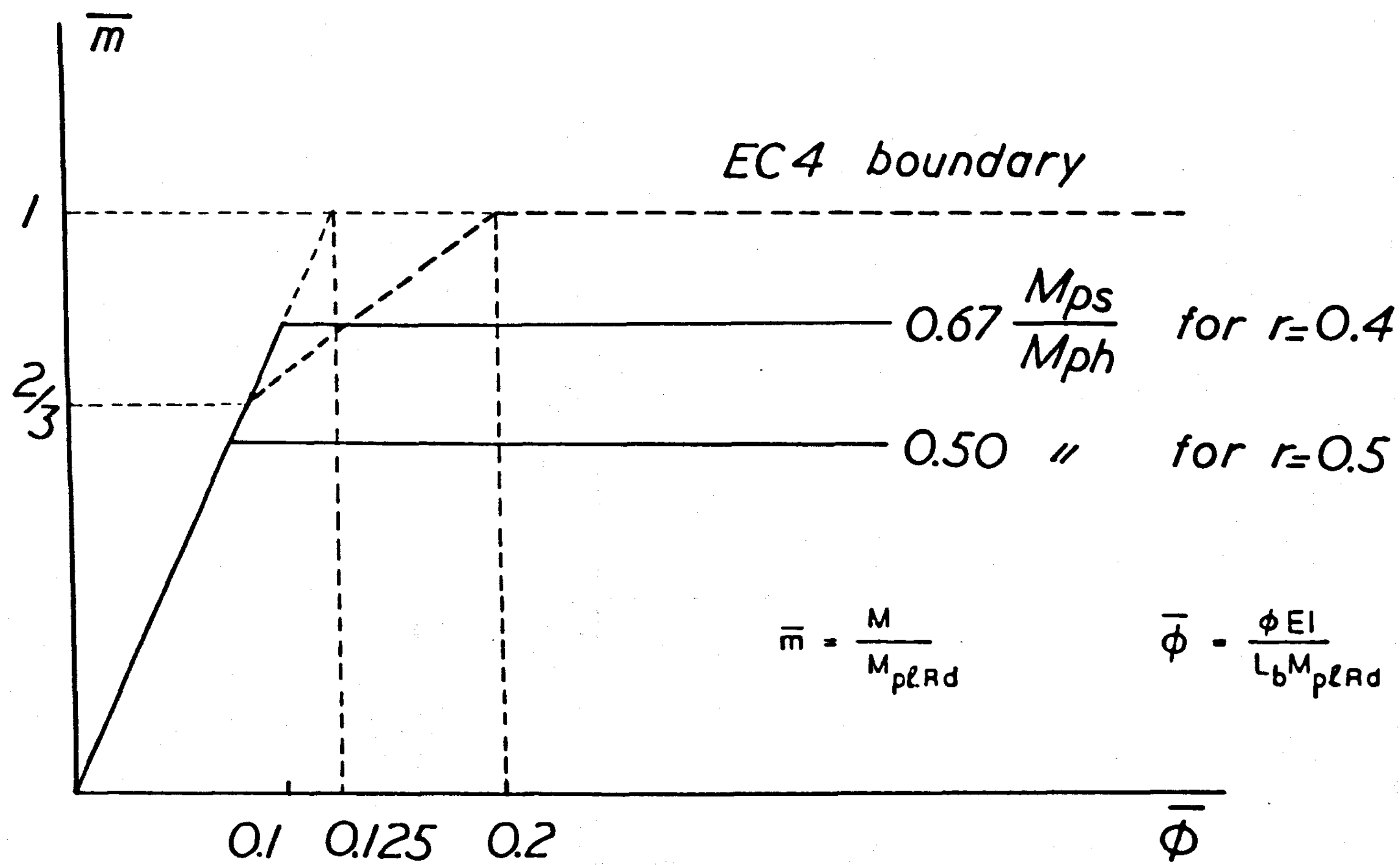


Fig. 8-9, Behaviour of end connections used in 8-4-3

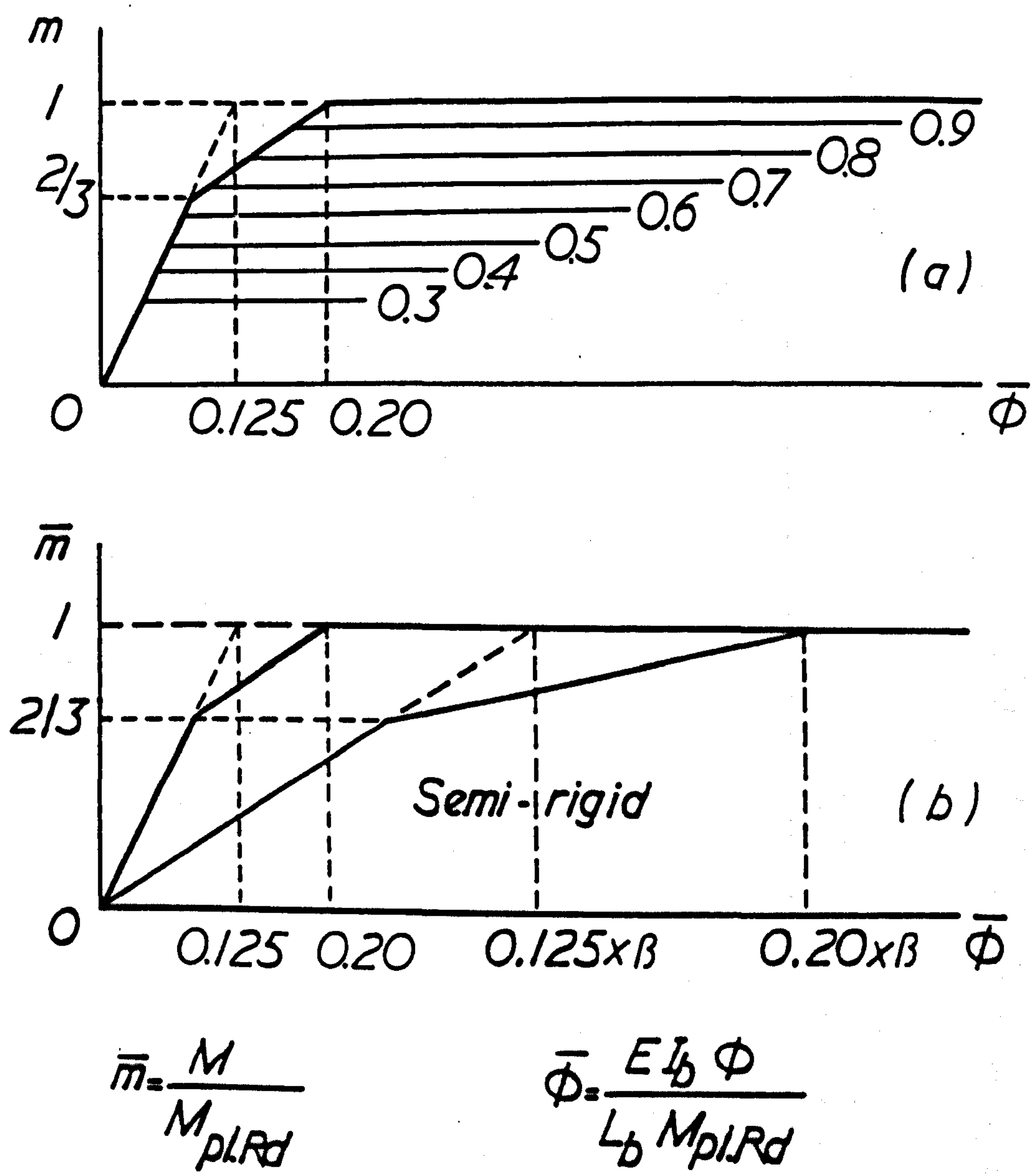


Fig. 8-10, The "standard procedure" used in the studies on required rotation

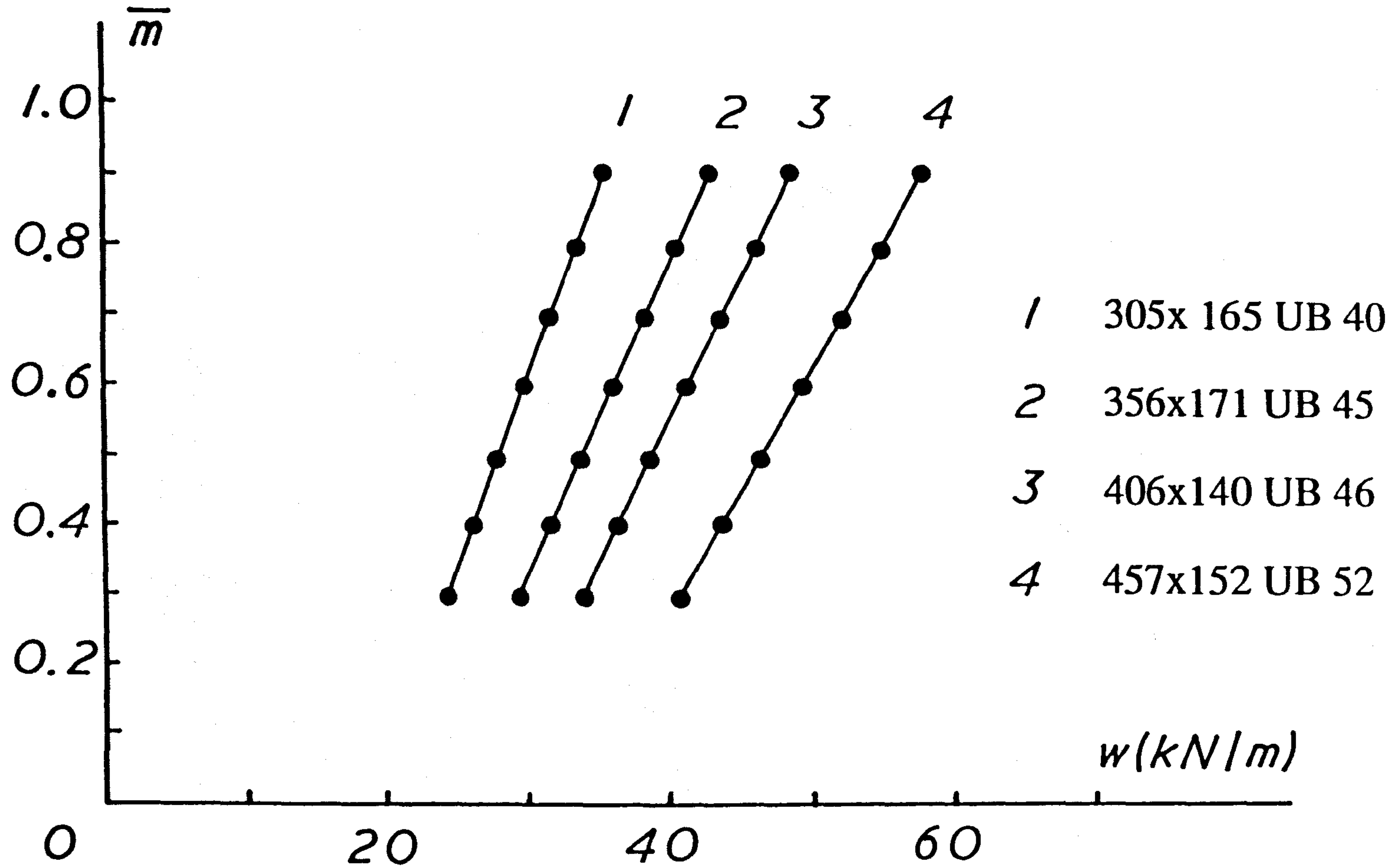


Fig. 8-11, Load factor at collapse versus assumed $\bar{m}$ of the connection for the beam with 11.5 m span

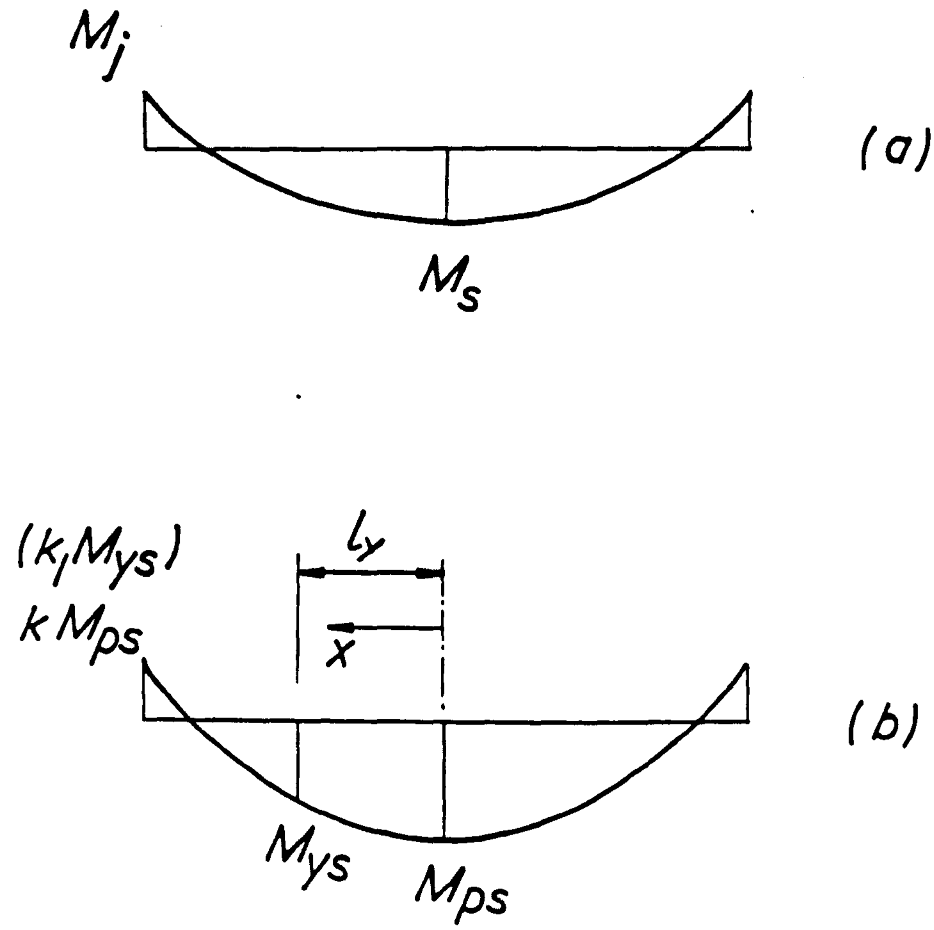


Fig. 8-12, Bending moment diagram of composite beam for elastic and plastic rotations

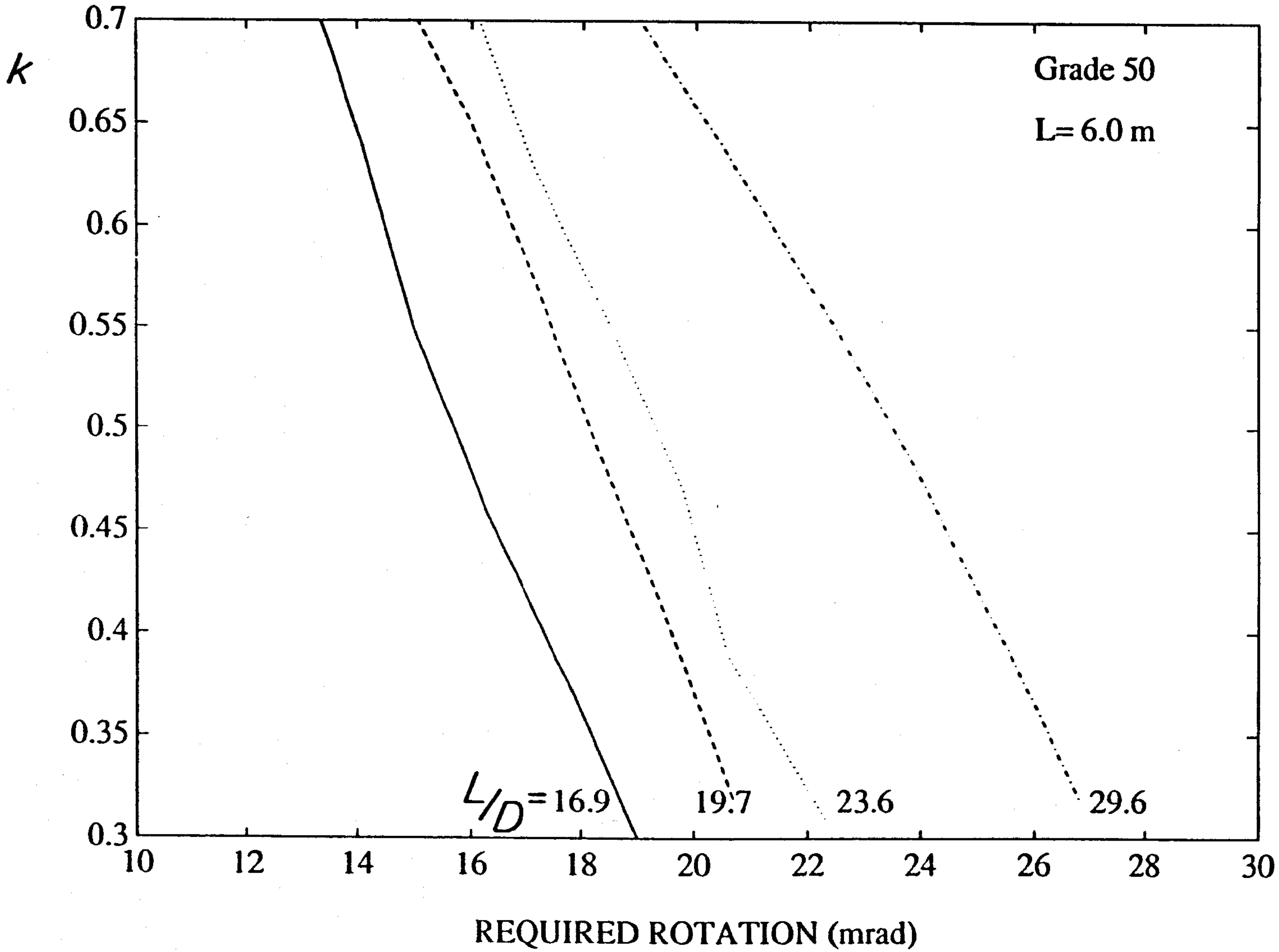


Fig. 8-13, Required rotation against ratio of connection moment to midspan moment for various span to depth ratios (6.0 m span, Grade 50 steel)

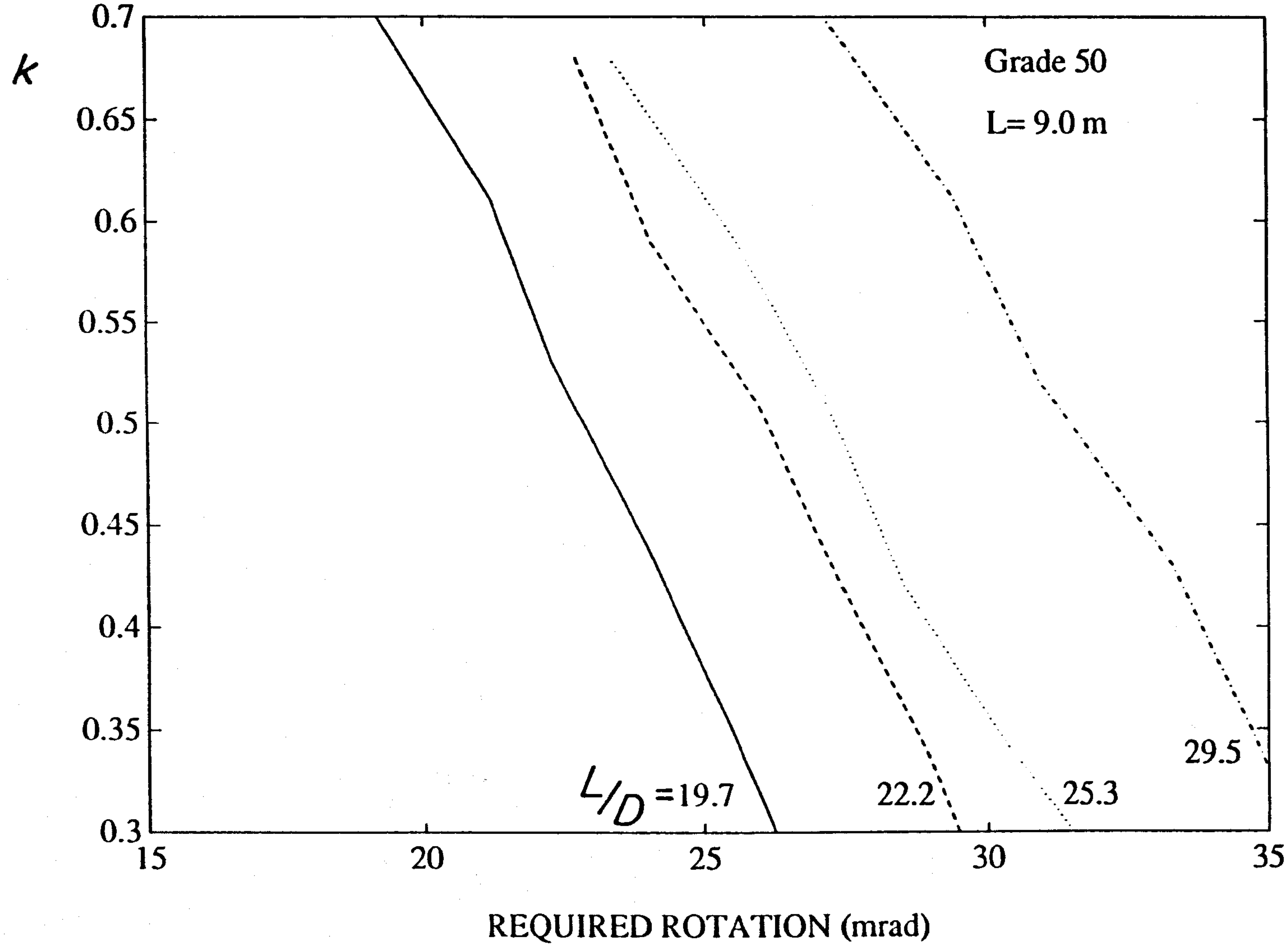


Fig. 8-14, Required rotation against ratio of connection moment to midspan moment for various span to depth ratios (9.0 m span, Grade 50 steel)

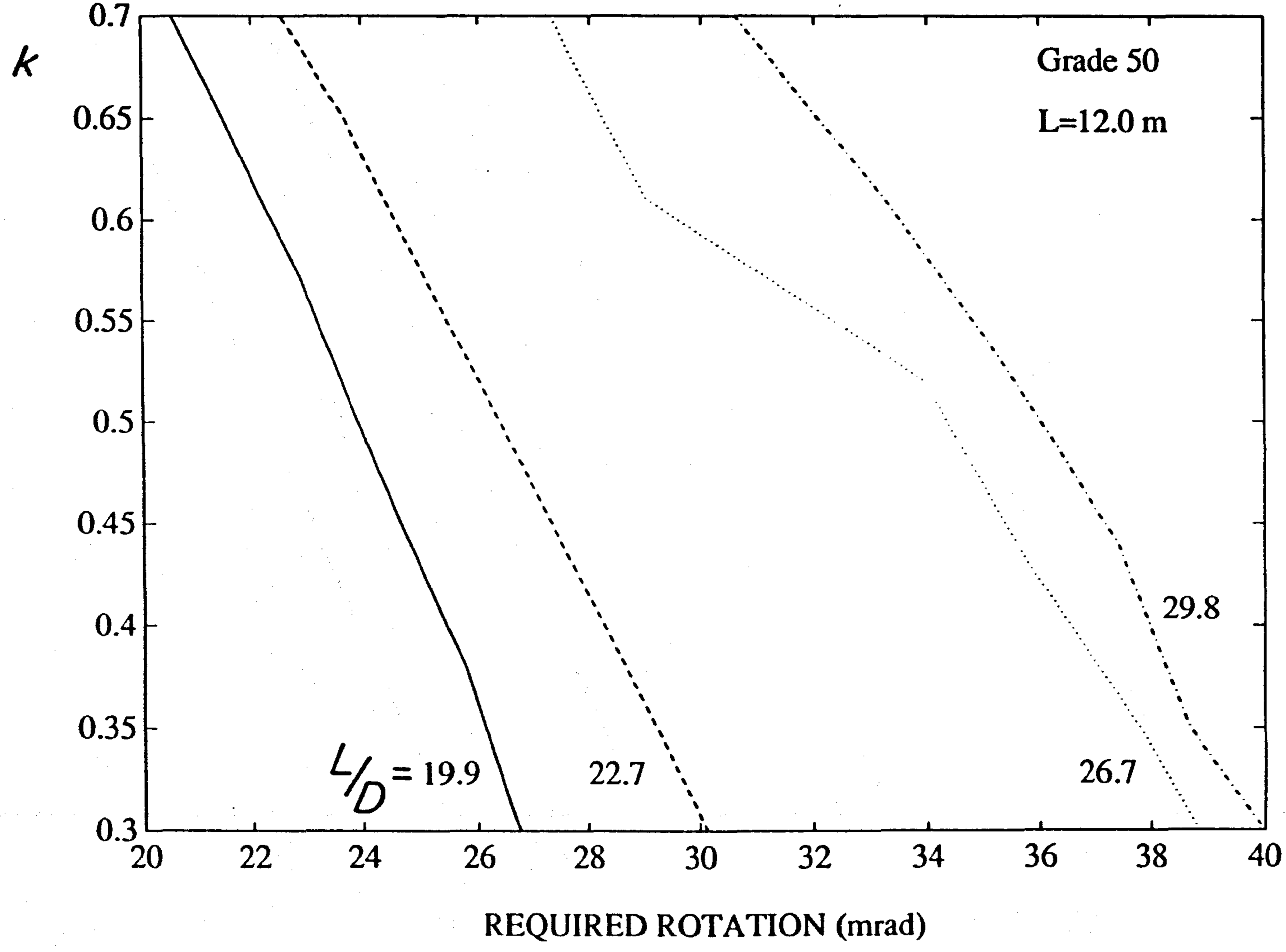


Fig. 8-15, Required rotation against ratio of connection moment to midspan moment for various span to depth ratios (12.0 m span, Grade 50 steel)

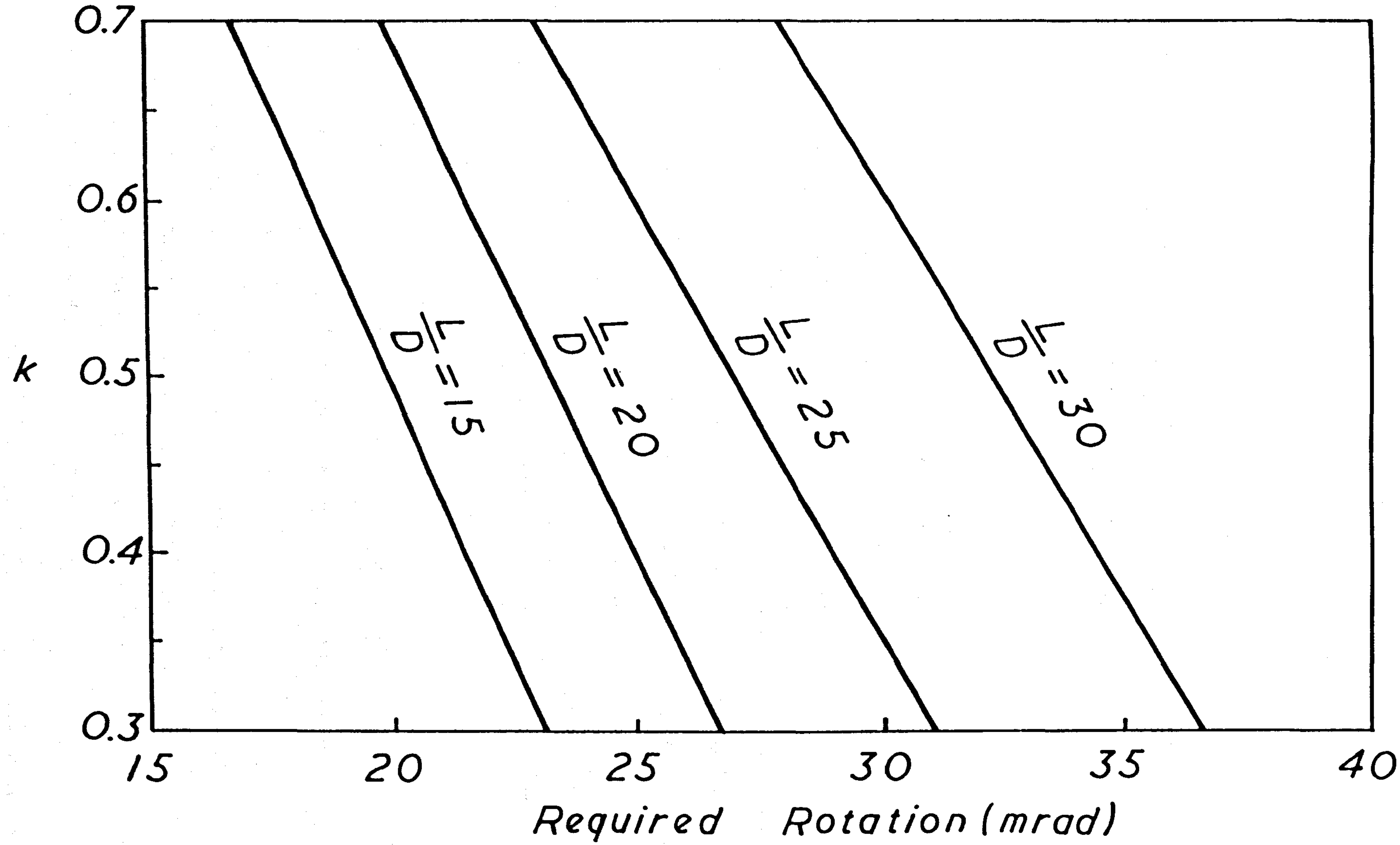


Fig. 8-16, Sample chart for required rotation against ratio of connection moment to midspan moment for various span to depth ratios

Chapter 9

CONCLUSIONS AND SUGGESTIONS FOR FURTHER WORK

This thesis has investigated the behaviour of end plate connections in steel and composite structures. The response of semi-rigid steel frames at serviceability and ultimate states has been examined. Experimental and theoretical studies have been undertaken on the behaviour of composite connections incorporating a concrete slab with profiled steel sheeting and an end plate steelwork joint. A method is proposed for prediction of the stiffness of composite connections. By developing a computer program, the subjects of redistribution of moments in semi-rigid composite beams and the required rotation capacity of connections for a full plastic mechanism have been studied. The following conclusions and suggestions for further work may be drawn from these investigations.

9-1-Conclusions of Study on Unbraced Semi-Rigid Steel Frames

Semi-rigid analysis of frames gives greater horizontal deflections and lower collapse loads in comparison with rigid analysis.

The classification method of EC3 for unbraced frames, used to differentiate rigid and semi-rigid connections, classifies most practical steel connections as semi-rigid. The EC3 prediction equation for the stiffness of end plate connections underestimates both the moment resistance and stiffness of such connections.

The allowable sway ratio of 1/300 in BS5950:Pt 1 can be relaxed to 1/270 where semi-rigid analysis is carried out. For the same reason, the sway limit in EC3 as 1/500 can be relaxed to 1/450 for semi-rigid analysis. This is concluded from analysis of a number of frames comprising semi-rigid connections with a moment-rotation behaviour taken as the rigid boundary in the EC3 classification of steel joints.

Where the wind-moment method is used in design, rigid deflections obtained from elastic analysis can be increased by 50% to allow for the connection flexibility at serviceability. This will decrease the need for rigorous analysis of such frames taking connection behaviour into account.

The approximated methods suggested by Wood & Roberts(1975) can be used to estimate the horizontal deflections of rigid frames. For semi-rigid frames, the method proposed by Driscoll(1976) gives sufficiently accurate results. In this method, the stiffnesses of the beams are reduced to allow for the flexibility of joints. The reduced stiffness of the beam is termed "relative stiffness". Where end connections are different in terms of stiffness, the "equivalent stiffness" can be assumed for the beam by simply averaging the stiffness of connections. This will result in acceptably accurate estimation of sway deflections. The "relative stiffness" and "equivalent stiffness" of beams can be used in the "substitute" frame (Wood & Roberts) to calculate the horizontal deflections.

The influence of the stiffness of semi-rigid joints on internal moments at collapse is less significant than its effect on sway at serviceability. Partial strength joints have a more significant influence on the collapse load than semi-rigid full strength joints.

9-2-Conclusions of Study on Composite Connections in Braced Frames

A comprehensive review has been made by the author on previous experimental works on composite connections. The available data have been summarized as a data base for future reference.

The experimental programme conducted by the author on cruciform specimens has been described. The instrumentation and testing procedures have been explained. The project consisted of a total of eleven tests, nine of which on composite connections and two on bare steel joints. The composite tests included six major axis and three minor axis connections, all comprising flush end plate steel joints (except one which was an extended end plate), profiled steel sheeting and stud shear connectors. The main variable was the amount of reinforcement, although the depth of steel section was also different in one major axis test. The bare steel tests consisted of a major axis and a minor axis flush end plate joint, similar to the steelwork joints in the composite tests.

Formulae have been given for prediction of the moment resistance of composite connections. These formulae can safely be used in design. A method has also been intoduced for determining the optimum amount of reinforcement in composite connections. In this method, the ratio of the force in the assumed reinforcement and the force in the bottom flange of steel beam is multiplied by a ratio in which the position of the joint's plastic neutral axis and the beam depth are taken into account. The resulting value is compared with unity. The assumed area of reinforcement is increased if this value is less than 1.0 and vice versa. This method provides a useful tool for an efficient design of composite joints.

From the test observations and the assessment of the results, the following conclusions may be drawn:

1) The amount of reinforcement increases directly the moment resistance of composite joints. Compared to steel joints, it is easier and more practical to provide full strength composite conections by simply increasing the amount of reinforcement.

2) The ductility of the overall reinforcement (rebars plus mesh) has a significant influence on the rotation capacity of composite connections. The beneficial effects of reinforcement are most significant for amounts of about 1% of the concrete area above the decking. The amount of reinforcement does not influence the initial stiffness of composite connections.

3) The increase in the depth of the steel beam will increase the stiffness and decrease the rotation capacity of a composite connection compared to a connection with

similar steelwork joint and a similar concrete slab having the same amount of reinforcement.

4) Minor axis connections (both steel and composite) are stiffer than major axis connections with similar connection components. The rotation capacity of the former is limited due to the rigidity of the steelwork joint.

5) All the tested composite connections have been classified as rigid according to EC4. The connections with 1% or more reinforcement are full strength while those with less than 1% reinforcement are partial strength. The classification of steel connections given by EC3 results in the tested major axis steel joint being semi-rigid and partial strength. The tested minor axis joint may be classified as rigid but still partial strength.

6) For steel flush end plate connections stiffened only in the compression zone, it is proposed to use the prediction equation of Frye & Morris(1975) for stiffened connections with the value of d taken as the lever arm between the tensile force in the top bolt row and the compressive force in the bottom flange of the beam.

7) The effect of semi-rigid connections on column stability has been studied. The results of the minor axis tests have been used in determining the effective length of the column. The change in unloading stiffness was found to be of little influence on the buckling length of columns. A value of 0.75 taken as the effective length ratio of columns in non-sway frames is safe and sufficiently accurate.

A method has been proposed for prediction of stiffness of composite connections. This is based on a mechanical model which consists of several springs, each simulating a source of stiffness. The contributions of the steelwork joint, reinforcement, shear connectors and concrete have been taken into account. A bilinear presentation of connection's moment-rotation curve has been suggested. This has a key point corresponding to the plastic design moment resistance of the composite connection calculated from the proposed formula (mentioned above), and the associated rotation determined from the relevant expression given in Chapter 7. The resulting $M-\phi$ curves are in good agreement with the experimental curves obtained in the author's tests. It underestimates slightly the resistance moment and the initial stiffness of the composite joint.

The following conclusions may be drawn from the studies on redistibution of moments in composite beams:

1) The redistribution of moment and the required rotation capacity for the end connections of a composite beam are interrelated. More redistribution of moments in composite beams necessitates more rotation capacity for the end connections.

2) The maximum percentage of redistribution of moments given as 40% in EC4 and BS5950:Pt 3.1 is an appropriate value for both propped and unpropped composite beams. Although more redistribution of moments may be possible in conservatively designed beams, values greater than 40% are not advisable.

3) Greater percentages of redistribution than those given by EC4 and BS5950:Pt 3.1 may be assumed for non-Plastic composite sections if further justification is provided. This is based on the satisfactory behaviour of Class 3 composite sections subjected to negative moment in the author's tests.

The subject of required rotation capacity of composite connections at the beam ends has been investigated. Methods have been proposed to determine the rotation capacity required for end connections in order to develop a full plastic mechanism. In this approach, the elastic and plastic rotations of the composite beam are calculated and added to find the required rotation of the joint. Both propped and unpropped construction have been considered. The expressions proposed by the author give generally safe and sufficiently accurate estimations of the required rotation. Simplified charts may be prepared for determination of rotation capacity of composite connections in order to achieve a full plastic mechanism in the beam.

The following can be deduced from the studies on the required rotation capacity for a complete plastic mechanism:

1) Generally, greater spans require more rotation capacity. Adopting a greater steel section for the composite beam of a specified span will reduce the required rotation capacity of the end connections.

2) It is possible to reduce the rotation requirements of connections by not utilizing the full sagging moment resistance of the composite beam.

3) The flexibility of partial strength connections does not influence the rotation needed for a plastic mechanism, rather its resistance determines the required rotation. Very flexible connections may cause the first hinge to form in midspan.

9-3-Suggestions for Further Work

The effect of semi-rigid connections on steel frames at serviceability and ultimate state has been considered in this thesis. The following suggestions are proposed for further research on the behaviour of semi-rigid steel frames:

1) The stiffness provided by cladding plays a considerable role in limiting the horizontal deflection. Although some guidance has been given on the prediction of cladding stiffness by Wood & Roberts(1975) and recently by Cunningham(1990), the absense of precise information about the effect of each type of cladding requires to be conservative about the overall stiffness of steel structures. To achieve more economy in semi-rigid design of unbraced steel frames, the effect of cladding should be included in the analytical studies.

2) The basic frames used in the studies on the effect of semi-rigid joints on the response of frames and the use of the approximate methods were all in the range of medium rise frames. Higher frames are needed to be checked for limiting sway and estimation of horizontal deflection.

3) EC3 states that first-order analysis may be employed for sway frames if columns are designed using an effective length greater than the real length or if the sway moments of the beams and connections are increased by an amplification factor. This needs to be examined for frames with semi-rigid connections.

The experimental studies of composite connections reported in this thesis concentrated on the influence of certain variables on the rotational behaviour of composite joints. The effect of the following are still needed to be investigated:

1) The degree of shear interaction between concrete slab and steel beam.

2) Different beam sections with variable depth and slenderness ratios of web and flanges, associated with adjusted reinforcement according to the proposed method for adopting an efficient amount of reinforcement, to check the general applicability of the method; also to check the behaviour of non-Plastic sections in hogging regions.

3) Providing a failure mode other than bolts in tension by using greater bolt size or more bolt rows.

4) Use of low amounts of longitudinal reinforcement with a stiffer connection, namely 0.5% reinforcement and mesh only with extended end plate connection, to study the possible achievement of proper moment resistance and rotation capacity with less reinforcement.

5) Stiffened column in both tension and compression zones, as well as an unstiffened column in the both regions.

6) The amount of transverse reinforcement in relation to the amount of longitudinal reinforcement.

In addition, further tests are proposed on cantilever specimens simulating external joints. Experimental study is suggested on the behaviour of external minor axis joints with especial attention to quantifying the restraint provided by the composite connection to the steel column.

The work described in Chapter 7 aimed at introducing the main idea of simulating composite connections as an idealized mechanical model. In this approach, the behaviour of materials was assumed linear elastic-plastic. The behaviour of connection elements can be assumed non-linear elasto-plastic as in reality. The general approach will still be applicable and may even produce better results. For instance, the non-linear behaviour of shear connectors can be introduced by using the expressions given by researchers for the load-slip behaviour. The method is particularly suitable for a computer program for determination of connection stiffness. A method is required for prediction of rotation capacity of composite connection.

The numerical studies in Chapter 8 were undertaken by use of a computer program for analysis of a single span composite beam with semi-rigid connections. The analysis ends when plastic hinges form in the beam rather than the connection. The post-buckling behaviour of the composite section in both hogging and sagging bending can be included in the program. The program can also be extended for a continuous composite beam, satisfying the compatibility conditions at the supports. Alternatively, the effect of adjacent spans may be taken into account by modifying the stiffness of end connections of a single span beam.

The semi-empirical formulae given in Chapter 8 for calculation of required rotation of connection for a full plastic mechanism have been based on limited available data. They have also been checked against the rigorous analysis of a composite beam with 1% reinforcement in the hogging regions and an assumed connection behaviour, subjected to uniformly distributed load. The effects of alteration in these parameters needs determining. Maximum span lengths may be defined for the varying parameters and redistribution percentages. Serviceability performance of composite beams with semi-rigid connections should also be considered.

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Appendix A

Design of Connections to Eurocode 3

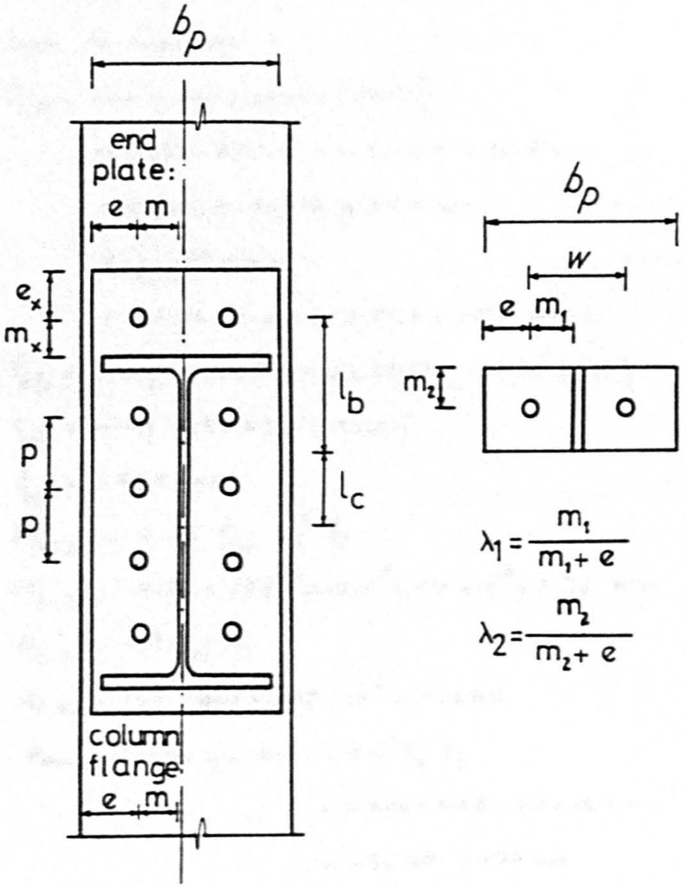
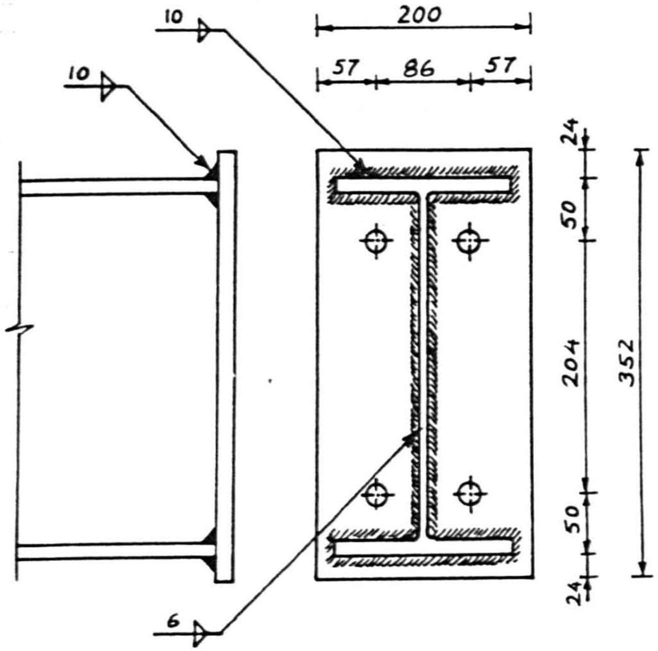


Fig. A-1, Notation used in the calculations of moment resistance of steel end plate joints according to Annex J of Eurocode 3

Remarks



BEAM	305x165 UB 40	GRADE 43	p_y = 310 N/mm^2
COLUMN	203 x 203 UC52	GRADE 43	p_y = 310 N/mm^2
END-PLATE	t_p = 15 mm	GRADE 43	p_y = 320 N/mm^2
BOLTS	M20	GRADE 8.8	
WELDS	FILLET WELDS	FLANGE WELD THROAT THICKNESS = 7mm	
		WEB WELD THROAT THICKNESS = 4 mm	

A-1

Remarks

J.3.4.1 Column Flange in tension

As only one bolt row is in tension, there are end-bolts to consider:

$\ell_{eff} = \min\{(4m+1.25e), (2\pi m)\}$

$m = (86-8.0)/2 - 0.8 \times 10.2 = 30.8$ mm

$e = (203.9-86)/2 = 59.0$ mm

$e_{min} = 57.0$ mm

$n = 1.25\,m = 1.25 \times 30.8 = 38.5$ mm

$\ell_{eff} = \min\{(4 \times 30.8 + 1.25 \times 59.0), (2 \times \pi \times 30.8)\}$

$\ell_{eff} = \min\{(197.0), (193.5)\}$

$\ell_{eff} = 193.5$ mm

$M_{Pl.Rd} = 0.25\, \ell_{eff}\, t_f^2\, f_y$

$M_{Pl.Rd} = 0.25 \times 193.5 \times 12.5^2 \times 310 \times 10^{-6} = 2.34$ kNm

$B_{t.Rd} = 0.9\, f_{ub}\, A_s$

$B_{t.Rd} = 0.9 \times 800 \times 245 \times 10^{-3} = 176$ kN

6.5.5(4) Punching shear per bolt $= \pi d^{\circ} \tau_y t_f$

$= \pi \times 40 \times 0.58 \times 310 \times 12.5 \times 10^{-3}$

$= 282$ kN > 176 kN

$\therefore$ not determining

mode 1: $F_{t.Rd} = 4 \times 2.34 \times 10^3 / 30.8 = 304$ kN

mode 2: $F_{t.Rd} = (2 \times 2.34 \times 10^3 + 2 \times 176 \times 38.5)/(30.8+38.5) = \boxed{263 \text{ kN}}$

mode 3: $F_{t.Rd} = 2 \times 176 = 352$ kN

Remarks

J.3.4.4 End-Plate in Tension

$\ell_{eff} = \min\{(\alpha m), (2\pi m)\}$

$m_1 = (86-6.1)/2 - 0.8 \times 4\sqrt{2} = 35.4$ mm

$m_2 = 50 - 10.2 - 0.8 \times 7\sqrt{2} = 31.8$ mm

$e = 57.0$ mm

$\lambda_1 = \dfrac{m_1}{m_1+e} = 0.38$

$\lambda_2 = \dfrac{m_2}{m_2+e} = 0.36$

Fig. J.3.7 $\therefore \alpha = 2\pi$

$\ell_{eff} = 2\pi m = 2 \times \pi \times 35.4 = 222.4$ mm

$n = 1.25 \times m_1 = 44.2$ mm

$M_{Pl.Rd} = 0.25 \times 222.4 \times 15^2 \times 320 \times 10^{-6} = 4.00$ kNm

mode 1: $F_{t.Rd} = 4 \times 4.00 \times 10^3 / 35.4 = 452$ kN

mode 2: $F_{t.Rd} = (2 \times 4.00 \times 10^3 + 2 \times 44.2 \times 176)/(35.4+44.2) = \boxed{296 \text{ kN}}$

mode 3: $F_{t.Rd} = 2 \times 176 = 352$ kN

A-2

J.3.4.5 Combination of End-Plate and Column Flange in Tension

Proc. J.3.1 step 3 — No distribution of forces is possible since only one bolt row exists in tension.

Proc. J.3.1 step 4 — $1.8\, B_{t.Rd} = 1.8 \times 176 = 317$ kN > 296 kN > 263 kN

$\therefore$ No need for change in the connection details.

		Remarks
J.3.4.6	Column Web in Tension	
	$F_{t.Rd} = f_{yc} \cdot t_{wc} \cdot b_{eff}$	
	$b_{eff} = \min\{(4m+1.25e),(2\pi m)\}$	
	$m = 30.8$ mm	
	$e = 59.0$ mm	
	$b_{eff} = \min\{(4\times30.8+1.25\times59.0),(2\times\pi\times30.8)\}$	
	$b_{eff} = \min\{(197.0),(193.5)\}$	
	$b_{eff} = 193.5$ mm	
	$F_{t.Rd} = 310\times8.0\times193.5\times10^{-3} = 480$ kN	$\boxed{480\,kN}$

		Remarks
J.3.5	Column Web in Compression	
	– Unstiffened Column Web	
	$F_{c.Rd} = f_{yc}\, t_{wc}\,(1.25-0.5\,\sigma_n/f_{yc})\, b_{eff} \leq f_{yc}\, t_{wc}\, b_{eff}$	
	σ_n = max. comp. normal stress in the web of the column due to axial force and bending	
	$\sigma_n = 0$	
	$b_{eff} = t_{fb} + 2\sqrt{2}\, a_e + 2t_e + 5(t_{fc}+r_c)$	
	$t_{fb} = 10.2$ mm	
	$a_e = 7$ mm	
	$t_e = 15$ mm	
	$t_{fc} = 12.5$ mm	
	$r_c = 10.2$ mm	A-3
	$b_{eff} = 10.2 + 2\sqrt{2}\times7 + 2\times15 + 5(12.5+10.2)$	
	$b_{eff} = 173.5$ mm	
	$F_{c.Rd} = 310\times8.0\times173.5\times10^{-3} = 430$ kN	$\boxed{430\,kN}$
J.3.6	Column Web in Shear	
	– Unstiffened Column Web	
	$V_{Rd} = f_{yc}\, A_v/\sqrt{3} = f_{yc}\, t_{wc}\, h_c/\sqrt{3}$	
	$V_{Rd} = 310\times8.0\times206.2\times10^{-3}/\sqrt{3} = 295$ kN	if the connection is to be symmetrically loaded, not applicable.

		Remarks
5.5.1	Buckling of the column web in compression	
	$N_{b.Rd} = A \cdot f_y \cdot X$	
	$A = b_{eff} \cdot t_{wc}$	
5.7.5	$b_{eff} = \left[h^2 + s_s^2\right]^{1/2}$	
	$h = 206.2$ mm	
	$s_s = t_{fb} + 2\sqrt{2}\, a_p + 2t_p$	
	$s_s = 10.2 + 2\sqrt{2} \times 7 + 2 \times 15 = 60.2$ mm	
	$b_{eff} = \left[(206.2)^2 + (60.2)^2\right]^{1/2} = 214.8$ mm	
	$f_y = 310\ N/mm^2$	
	X = the reduction factor obtained from table 5.5.2	
	$\bar{\lambda} = \dfrac{\lambda\sqrt{\beta_A}}{\lambda_1}$	
	$\beta_A = 1$ for class 1, 2 & 3 sections	
	$\lambda_1 = 93.9\sqrt{\dfrac{235}{f_y}} = 82$	
	$\lambda = \dfrac{l_e}{i} = \dfrac{2.5\, d_c}{t_{wc}} = \dfrac{2.5 \times 160.8}{8.0} = 50$	
	$\bar{\lambda} = \dfrac{50\sqrt{1}}{82} = 0.61$	
table 5.5.3	Selection of buckling curve for determination of X:	
	$\dfrac{h}{b} < 1.2$, $t_f < 100$ mm	
	buckling about 2-2 axis ∴ Curve "c"	
	$X = 0.7912$	
	$N_{b.Rd} = 214.8 \times 8.0 \times 310 \times 0.7912 \times 10^{-3} = 421$ kN	$\boxed{421\ kN}$

		Remarks
	Summary of Design Resistances of Critical Zones :	
Sheet No 2	Column Flange in tension — 263 kN	
Sheet No 3	End-plate in tension — 296 kN	
Sheet No 4	Column web in tension — 480 kN	
Sheet Nos 5&6	Column web in compression — 430 kN , 421 kN	
	∴ Column flange in tension needs to be stiffened to develop the connection strength. Being used in a composite connection , it would be improved by the composite action of tensile zone.	
	∴ No need for column web in compression to be stiffened, but for the sake of similarity to the extended end-plate connection , let it be stiffened. Reference should be made to the calculations on extended end-plate connection, particularly sheet No 9 .	A-4
J.3.5.2 (1)	Stiffened column	
J.2.3.3 (1)	Use plate 12 mm thick both sides	
	$f_y = 320\ N/mm^2$,	
	Fillet weld both sides of each plate $a = 5$ mm	

		Remarks
Proc. J.3.1	Check the beam cross section adjacent to the end-plate	

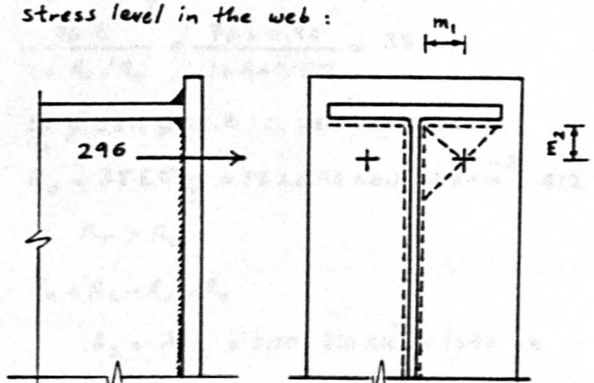
Stress level in the tension or compression flange :

$$\sigma_{ft} = \frac{F_{t.RD}}{t_{fb} \cdot b_{fb}}$$

$$\sigma_{ft} = \frac{263 \times 10^3}{10.2 \times 165.1} = 156\ N/mm^2 < 310\ N/mm^2$$

(The collaboration of the web is ignored, see sheet No 9)

• step 14 — stress level in the web :



$m_1 = 35.4$ mm

$m_2 = 32.0$ mm

sheet No 3 — force in the bolt row = 296 kN

— as an alternative based on the maximum possible tension of top flange ∴ $\frac{263 \times (303.8 - 10.2)}{204 + 50 + 10.2/2} = 298$ kN

— the proportion to be resisted by the beam web = F_{wt}

$$F_{wt} = 298 \times \frac{32.0}{32.0 + 35.4} = 141\ kN$$

$$\sigma_{wt} = \frac{F_{wt}}{t_w\,(m_1 + m_2)} = \frac{141 \times 10^3}{6.1\,(35.4 + 32.0)} = 343\ N/mm^2$$

∴ stress level at the beam web in tension is higher than the yield stress (310 N/mm^2). Note that shear is not yet considered.

		Remarks
6.6.5.3	Fillet welds connecting the beam web to the end-plate	

$$f_{w.Rd} = f_{vw} \cdot a$$

$$f_{vw} = \frac{f_u/\sqrt{3}}{\beta_w\, \gamma_{Mw}} \qquad \gamma_{Mw} = 1$$

f_u = ultimate tensile strength of the weaker part

$f_{u\,steel} = 465\ N/mm^2$ — $f_{uw} = 450\ N/mm^2$

6.6.5.3(5) — $\beta_w = 0.85$

$$f_{vw} = \frac{450\sqrt{3}}{0.85} = 306\ N/mm^2$$

$$f_{w.Rd} = f_{vw} \cdot a = 306 \times 4 = 1224\ N/mm$$

total weld strength in tension $= 1224 \times 2 \times (35.4 + 32.0)\,10^{-3}$

braced frame — $= 165\ kN < 141\ kN \times 1.4$

The weld strength is less than factored force but more than the unfactored force. If any sign of failure was not found in the full scale test, it could be said that either applying the factor of 1.4 is conservative, or the preceding calculation of the tension force in the web, overstimates the web proportion. If the latter is the case, the omition of web collaboration in tension becomes reasonable.

Fillet welds connecting the beam flange to the end-plate

Effective length of weld = $2 \times 165.1 = 230.2$ mm

total weld strength in tension $= 306 \times 7 \times 230.2 \times 10^{-3} = 493$ kN

J.3.4.4(6) braced frame — 1.4 (design moment resistance of the connection) $= 1.4 \times 263 \times (303.8 - 12.5) \times 10^{-3} = 107$ kN

design plastic moment resistance of the beam $= 310 \times 624.5 \times 10^{-3} = 194$ kNm

∴ 493 kN > 1.4 × 263 kN

A-5

		Remarks

Plastic Moment Resistance of Composite Section (Rebars only)

B.2.4
BS5950 (3-1)

$\frac{d}{t} = \frac{265.6}{6.1} = 43.6$

$\varepsilon = \sqrt{\frac{275}{f_y}} = \sqrt{\frac{275}{310}} = 0.94$

$38\varepsilon = 38 \times 0.94 = 35.8$

$R_r = 486 \times 905 \times 10^{-3} = 440$ kN

$R_v = d t p_y = 265.6 \times 6.1 \times 310 \times 10^{-3} = 502$ kN

$\frac{76\varepsilon}{1 + R_r/R_v} = \frac{76 \times 0.94}{1 + 440/502} = 38.1$

$\frac{d}{t} > 38.1 > 35.8$ ∴ web not compact

$R_o = 38 \varepsilon t^2 p_y = 38 \times 0.94 \times 6.1^2 \times 310 \times 10^{-3} = 412$ kN

$\therefore R_r > R_o$

$R_n = R_s - R_v + R_o$

$R_s = A_s p_y = 5150 \times 310 \times 10^{-3} = 1597$ kN

$R_n = 1597 - 502 + 412 = 1507$ kN

$\therefore R_r < R_n$ Plastic neutral axis in steel flange

$M_c = R_n \frac{D}{2} + R_r D_r - \frac{(R_n - R_r)^2}{R_f} \frac{T}{4}$ $D_r = 90$ mm

$M_c = 1507 \times \frac{303.8}{2} + 440 \times 90 - \frac{(1507 - 440)^2}{522} \times \frac{10.2}{4}$

$(R_f = BTp_y = 165.1 \times 10.2 \times 310 \times 10^{-3} = 522$ kN$)$

$M_c = 263$ kNm — $\boxed{263 \text{ kNm}}$

		Remarks

Plastic Moment Resistance of Composite Section (Mesh + Rebars)

8Φ12 — $F_r = 486 \times 905 \times 10^{-3} = 440$ kN

6Φ6 — $F_{mesh} = 668 \times 162 \times 10^{-3} = 108$ kN

$R_r = 440 + 108 = 548$ kN

$D_r = \frac{440 \times 90 + 108 \times 87}{548} = 89.4$ mm

sheet No 10 — $R_v = d t p_y = 502$ kN

$38\varepsilon = 38\sqrt{\frac{275}{310}} = 38 \times 0.94 = 35.8$

$\frac{76\varepsilon}{1 + R_r/R_v} = \frac{76 \times 0.94}{1 + 548/502} = 34.2$

$\frac{d}{t} = 43.6 > 35.8 > 34.2$ ∴ web not compact

$R_o = 38 \varepsilon t^2 p_y = 38 \times 0.94 \times 6.1^2 \times 310 \times 10^{-3} = 412$ kN

$\therefore R_r > R_o$

$R_n = R_s - R_v + R_o$

$R_s = A_s p_y = 5150 \times 310 \times 10^{-3} = 1597$ kN

$R_n = 1597 - 502 + 412 = 1507$ kN

$\therefore R_r < R_n$ Plastic neutral axis in steel flange

$M_c = R_n \frac{D}{2} + R_r D_r - \frac{(R_n - R_r)^2}{R_f} \frac{T}{4}$

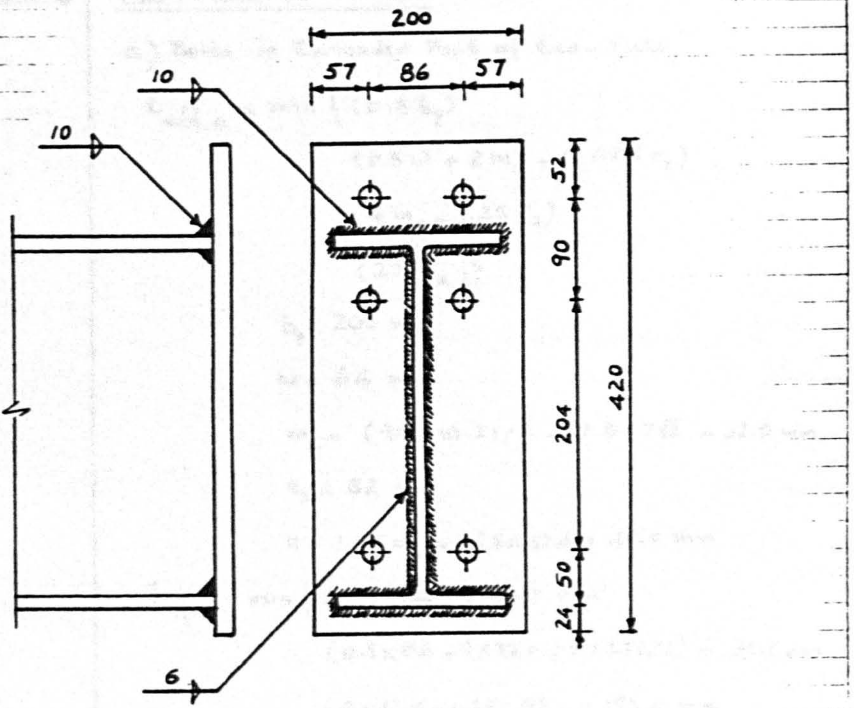
$M_c = 1507 \times \frac{303.8}{2} + 548 \times 89.4 - \frac{(1507 - 548)^2}{522} \times \frac{10.2}{4}$

$(R_f = BTp_y = 165.1 \times 10.2 \times 310 \times 10^{-3} = 522$ kN$)$

$M_c = 273$ kNm — $\boxed{273 \text{ kNm}}$

A-6

Remarks



BEAM	305 x 165 UB 40	GRADE 43	$P_y = 310\ N/mm^2$
COLUMN	203 x 203 UC 52	GRADE 43	$P_y = 310\ N/mm^2$
END-PLATE	$t_p = 15$ mm	GRADE 43	$P_y = 320\ N/mm^2$
BOLTS	M20	GRADE 8.8	
WELDS	FILLET WELDS	FLANGE WELD THROAT THICKNESS = 7 mm	
		WEB WELD THROAT THICKNESS = 4 mm	

Remarks

J.3.4.1

Column Flange in Tension

$$l_{eff} = \min \{ (0.5p + 2m + 0.625e),\ (4m + 1.25e),\ (2\pi m) \}$$

$p = 90$ mm

$m = (86 - 8)/2 - 0.8 \times 10.2 = 30.8$ mm

$e = (203.9 - 86)/2 = 59.0$ mm

$e_{min} = 57.0$ mm

$n = 1.25\, m = 1.25 \times 30.8 = 38.5$ mm

$$l_{eff} = \min \{ (0.5 \times 90.0 + 2 \times 30.8 + 0.625 \times 59.0) = 143.5\ mm,\ (4 \times 30.8 + 1.25 \times 59.0) = 197.0\ mm,\ (2 \times \pi \times 30.8) = 193.5\ mm \}$$

A-7

$l_{eff} = 143.5$ mm

$M_{Pl.Rd} = 0.25\, l_{eff}\, t_f^2\, f_y$

$M_{Pl.Rd} = 0.25 \times 143.5 \times 12.5^2 \times 310 \times 2 \times 10^{-6} = 3.48$ kNm

$B_{t.Rd} = 0.9\, f_{ub}\, A_s$

$B_{t.Rd} = 0.9 \times 800 \times 245 \times 10^{-3} = 176.0$ kN

mode 1: $F_{t.Rd} = 4 \times 3.48 \times 10^3 / 30.8 = 452$ kN $\boxed{452\ kN}$

mode 2: $F_{t.Rd} = (2 \times 3.48 \times 10^3 + 4 \times 38.5 \times 176)/(30.8 + 38.5) = 492$ kN

mode 3: $F_{t.Rd} = 4 \times 176 = 704$ kN

		Remarks
J.3.4.4	End – Plate in Tension	

a) Bolts in Extended Part of End-Plate

$$l_{eff.a} = \min\left\{ \begin{array}{l} (0.5\, b_p) \\ (0.5w + 2m_x + 0.625 e_x) \\ (4 m_x + 1.25 e_x) \\ (2\pi m_x) \end{array} \right\}$$

$b_p = 200$ mm

$w = 86$ mm

$m_x = (90 - 10.2)/2 - 0.8 \times 7\sqrt{2} = 32.0$ mm

$e_x = 52$ mm

$n = 1.25\, m_x = 1.25 \times 32.0 = 40.0$ mm

$$l_{eff.a} = \min\left\{ \begin{array}{l} (0.5 \times 200) = 100.0 \text{ mm} \\ (0.5 \times 86 + 2 \times 32.0 + 0.625 \times 52) = 139.5 \text{ mm} \\ (4 \times 32.0 + 1.25 \times 52) = 193.0 \text{ mm} \\ (2 \times \pi \times 32.0) = 201.0 \text{ mm} \end{array} \right\}$$

$l_{eff.a} = 100.0$ mm

$M_{Pl.Rd} = 2 \times 0.25 \times 100.0 \times 15^2 \times 320 \times 10^{-6} = 3.60$ kNm

mode 1 : $F_{t.Rd} = 1/2 \times 4 \times 3.60 \times 10^3 / 32.0 = 225$ kN — $\boxed{225 \text{ kN}}$

mode 2 : $F_{t.Rd} = 1/2\,(2 \times 3.60 \times 10^3 + 40.0 \times 4 \times 176)/(32.0 + 40.0) = 246$ kN

mode 3 : $F_{t.Rd} = 1/2 \times 4 \times 176 = 352$ kN

		Remarks

b) Bolts Row Below The Tension Flange

$l_{eff.b} = \min\{(\alpha m), (2\pi m)\}$

$m = (86 - 6.1)/2 - 0.8 \times 4\sqrt{2} = 35.4$ mm

$m_1 = 35.4$ mm

$m_2 = 32.0$ mm

$n = 40.0$ mm

$$\lambda_1 = \frac{35.4}{35.4 + 57} = 0.38$$

$$\lambda_2 = \frac{32.0}{32.0 + 57} = 0.36$$

Fig J.3.7 — $\alpha = 2\pi$

$l_{eff.b} = 2\pi \times 35.4 = 222.4$ mm

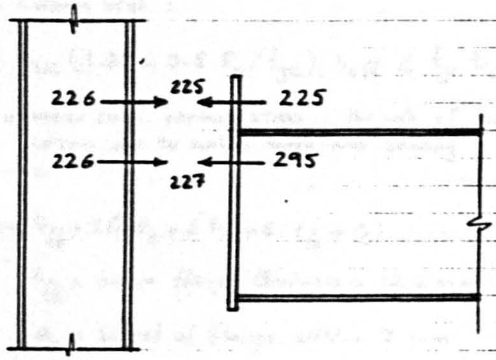
$M_{Pl.Rd} = 0.25 \times 222.4 \times 15^2 \times 320 \times 10^{-6} = 4.07$ kNm

mode 1 : $F_{t.Rd} = 4 \times 4.07 \times 10^3 / 35.4 = 460$ kN

mode 2 : $F_{t.Rd} = (2 \times 4.07 \times 10^3 + 2 \times 40.0 \times 176)/(35.4 + 40.0) = \boxed{295 \text{ kN}}$

mode 3 : $F_{t.Rd} = 2 \times 176 = 352$ kN

		Remarks
J.3.4.5	Combination of End-Plate and Column Flange in Tension	



total force in bolts = 520 kN

total force in column flange = 452

Proc. J.3.1 step 3 — No remarkable shifting of forces occurs.

total force in beam flange = 225 + 227 = 452 kN

Proc. J.3.1 step 4 — $1.8\, B_{t.Rd} = 1.8 \times 176 = 317$ kN > 295 kN > 225 kN

$\therefore$ No need for change in connection details.

		Remarks
J.3.4.6	Column Web in Tension	

$$F_{t.Rd} = f_{yc} \cdot t_{wc} \cdot b_{eff}$$

J.3.4.1(2)

$$b_{eff} = 2 \min \left\{ (0.5p + 2m + 0.625e),\ (4m + 1.25e),\ (2\pi m) \right\}$$

$p = 90$ mm

$m = 30.8$ mm

$e = 59.0$ mm

$$b_{eff} = 2 \min \left\{ \begin{array}{l} (0.5 \times 90 + 2 \times 30.8 + 0.625 \times 59.0) = 143.5 \text{ mm} \\ (4 \times 30.8 + 1.25 \times 59.0) = 197.0 \text{ mm} \\ (2 \times \pi \times 30.8) = 193.5 \text{ mm} \end{array} \right\}$$

$$b_{eff} = 2 \times 143.5 = 287.0 \text{ mm}$$

$$F_{t.Rd} = 310 \times 8.0 \times 287.0 \times 10^{-3} = 712 \text{ kN}$$

A-9

$\boxed{712\text{ kN}}$

		Remarks
J.3.5	Column Web in Compression	
J.3.5.1	– Unstiffened Column Web :	
	$F_{c.Rd} = f_{yc}\, t_{wc}\,(1.25 - 0.5\, \sigma_n / f_{yc})\, b_{eff} \leqslant f_{yc}\, t_{wc}\, b_{eff}$	
	σ_n = max. comp. normal stress in the web of the column due to axial force and bending	
	$\sigma_n = 0$	
	$b_{eff} = t_{fb} + 2\sqrt{2}\, a_e + 2\, t_e + 5(t_{fc} + r_c)$	
	t_{fb} = beam flange thickness = 10.2 mm	
	a_e = throat of flange weld = 7 mm	
	t_e = end-plate thickness = 15 mm	
	t_{fc} = column flange thickness = 12.5 mm	
	r_c = root radius of column = 10.2 mm	
	$b_{eff} = 10.2 + 2\sqrt{2} \times 7 + 2 \times 15 + 5(12.5 + 10.2)$	
	$b_{eff} = 173.5\,mm$	
	$F_{c.Rd} = 310 \times 8.0 \times 173.5 \times 10^{-3} = 430\ kN$	$\boxed{430\ kN}$
J.3.6	Column Web in Shear	
J.3.6.1	– Unstiffened Column Web :	
	$V_{Rd} = f_{yc}\, A_v / \sqrt{3} = f_{yc}\, t_{wc}\, h_c / \sqrt{3}$	
	$V_{Rd} = 310 \times 8.0 \times 206.2 \times 10^{-3} / \sqrt{3} = 295\ kN$	if the connection is to be symmetrically loaded, not applicable.

		Remarks
5.5.1	Buckling of the Column Web in Compression	
	$N_{b.Rd} = A \cdot f_y \cdot \chi$	
	$A = b_{eff} \cdot t_{wc}$	
5.7.5	$b_{eff} = [h^2 + S_s^2]^{1/2}$	
Fig. 5.7.2	$h = 206.2\ mm$	
"	$S_s = t_{fb} + 2\sqrt{2}\, a_p + 2\, t_p$	
	$S_s = 10.2 + 2\sqrt{2} \times 7 + 2 \times 15 = 60.2\ mm$	
	$b_{eff} = [(206.2)^2 + (60.2)^2]^{1/2} = 214.8\ mm$	
	$f_y = 310\ N/mm^2$	
	χ = the reduction factor obtained from table 5.5.2	
	$\bar{\lambda} = \dfrac{\lambda \sqrt{\beta_A}}{\lambda_1}$	
	$\beta_A = 1$ for class 1, 2 & 3 sections	
	$\lambda_1 = 93.9 \sqrt{\dfrac{235}{f_y}} = 82$	
	$\lambda = \dfrac{l_e}{r} = \dfrac{2.5\, d_c}{t_{wc}} = \dfrac{2.5 \times 160.8}{8.0} = 50$	
	$\bar{\lambda} = \dfrac{\lambda}{\lambda_1} [\beta_A]^{1/2} = \dfrac{50}{82} [1] = 0.61$	
table 5.5.3	Selection of buckling curve for determination of χ,	
	$h/b < 1.2$, $t_f < 100\ mm$	
Fig. J.2.4(a)	buckling about z-z axis ∴ curve "c"	
table 5.5.2	$\chi = 0.7912$	
	$N_{b.Rd} = 214.8 \times 8.0 \times 310 \times 0.7912 \times 10^{-3} = 421\ kN$	$\boxed{421\ kN}$

A-10

Remarks

Summary of Design Resistances of Critical Zones:

Sheet No 2	Column flange in tension	452 kN
Sheet No 5	End-plate in tension	452 kN
Sheet No 6	Column web in tension	712 kN
Sheet No 7,8	Column web in Compression	430 kN , 421 kN

∴ Column web in compression needs to be stiffened to develop the connection strength:

J.3.5.2(1) **Stiffened Column**

J.2.3.3(1) Use plate 12 mm thick both sides,

$f_y = 320\ N/mm^2$,

Fillet weld both sides of each plate $a = 5$ mm

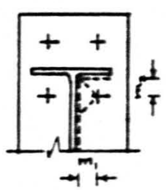
Check the beam cross section adjacent to the end-plate

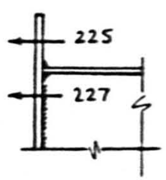
Stress level in the tension or compression flange:

$$\sigma_{ft} = \frac{F_{t.Rd}}{t_{fb} \cdot b_{fb}}$$

$$\sigma_{ft} = \frac{452 \times 10^3}{10.2 \times 165.1} = 268\ N/mm^2 < 310\ N/mm^2$$ (ignoring the collaboration of the web)

Stress level in the web:





$m_1 = 35.4$ mm

$m_2 = 32.0$ mm

Remarks

Sheet No 5 — force in the lowest bolt row = 227 kN

the proportion to be resisted by the beam web = F_{wt}

$$F_{wt} = 227 \times \frac{32.0}{35.4 + 32.0} = 108\ kN$$

$$\sigma_{wt} = \frac{F_{wt}}{t_{wb}(m_1 + m_2)} = \frac{108 \times 10^3}{6.1 \times (35.4 + 32.0)} = 263\ N/mm^2$$

∴ $\sigma_{wt} = 263\ N/mm^2 < 310\ N/mm^2$ but the interaction of shear and tension must be considered when the shear force of the connection is determined.

Design moment resistance of the connection

Sheet No 8 — $M_d = 452 \times (303.8 - 10.2) \times 10^{-3} = 133$ kN m

6.6.5.3 — **Fillet welds connecting the beam web to the end plate**

$F_{w.Rd} = f_{vw} \cdot a$

$$f_{vw} = \frac{f_u/\sqrt{3}}{\beta_w \gamma_{Mw}} \qquad \gamma_{Mw} = 1$$

f_u = ultimate tensile strength of the weaker part

$f_{u\,steel} = 465\ N/mm^2$ — $f_{uw} = 450\ N/mm^2$

6.6.5.3(5) — $\beta_w = 0.85$

$$f_{vw} = \frac{450/\sqrt{3}}{0.85} = 306\ N/mm^2$$

$F_{w.Rd} = 306 \times 4 = 1224\ N/mm$

total weld strength in tension $= 1224 \times 2 \times (35.4 + 32.0)\,10^{-3}$

braced frame — $= 165\ kN > 108\ kN \times 1.4$

Annex M(4) — The normal stress parallel to the axis of weld is not considered.

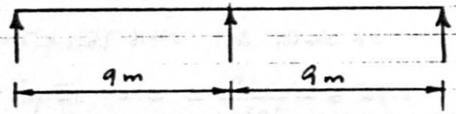
∴ shear force need not to be checked.

		Remarks
	<u>Fillet Welds Connecting the Beam Flange to the End-Plate</u>	
	Effective Length of Weld $\leq 2 \times 165.1 = 230.2$ mm	
	$f_{w.Rd} = f_{vw} \cdot a = 306 \times 7 = 2142$ N/mm	
	Total weld strength in tension $= 2142 \times 230.2 \times 10^{-3} = 493$ kN	
J.3.4.4 (6)	Design plastic moment of resistance of the beam $= 310 \times 624.5 \times 10^{-3} =$	194 kNm
Sheet No 10	1.4 (design plastic moment of the connection) $= 1.4 \times 133 = \underline{186}$ kNm	
Sheet No 9	Maximum tension in beam flange $= 452$ kN	
	$\therefore$ 493 kN $< 1.4 \times 452$ kN	
	But ductility required from weld was limited by onset of local	
	buckling in the beam.	

Appendix B

Design Calculations of Composite Beams to BS5950 Part 3.1

Remarks



Unpropped construction

Beams at 3 m centers , Uniform loading

Materials

Structural steel Grade 43	$p_y = 275\ N/mm^2$
	$E = 205\ kN/mm^2$
Concrete N.W. Grade 30	$f_{cu} = 30\ N/mm^2$
	$\gamma = 2300\ kg/m^3$
Deck PMF CF 46	$D_s = 120\ mm$
	$D_p = 46\ mm$
	$S = 225\ mm$
	$p_{yp} = 280\ N/mm^2$
	$t_p = 0.9\ mm$

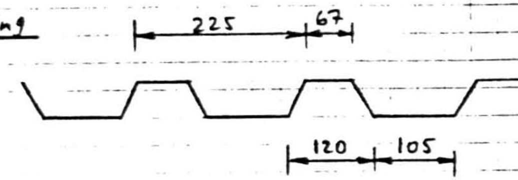
Studs 19 mm diameter × 95 mm height

Reinforcement	Tore steel	$p_y = 460\ N/mm^2$
Mesh A142		$f_y = 485\ N/mm^2$

Remarks

Floor loading

PMF manual



Assuming symmetric deck profile

average depth $= 120 - \frac{46}{2} = 97$ mm

weight of concrete $= 2300 \times 9.81 \times 97 \times 10^{-6} =$	$2.19\ kN/m^2$	
Steel deck	0.11 "	
Total	$2.30\ kN/m^2$	

Construction stage

	Floor	$2.30\ kN/m^2$	
	Steel beam	$0.15\ kN/m^2$	
	Dead load	$2.45\ kN/m^2$	$2.45\ kN/m^2$
2.2.3	Imposed construction load	0.50 "	$0.50\ kN/m^2$

Composite Stage

Floor + steel beam	$2.45\ kN/m^2$	
Ceiling	$0.55\ kN/m^2$	
Dead load	$3.00\ kN/m^2$	$3.00\ kN/m^2$
Imposed load	$5.00\ kN/m^2$	
Partitions	$1.00\ kN/m^2$	
Total Imposed load	$6.00\ kN/m^2$	$6.00\ kN/m^2$

B-1

Remarks

<u>Initial Selection of beam size</u>

Try 305 x 165 x 40 UB Grade 43

Span / depth ratio $= \frac{9000}{305} = 29.5$

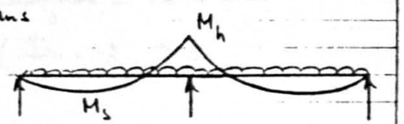
Pt 1, table 6 — $T = 10.2\ \text{mm} \not> 16\ \text{mm} \quad \therefore\ p_y = 275\ \text{N/mm}^2$

<u>Construction stage</u>

<u>ULS</u>

Design load $= (1.6 \times 0.5 + 1.4 \times 2.45) \times 3 = 12.7\ \text{kN/m}$

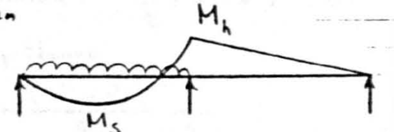
a) load on both spans



$M_h = 0.125 w l^2 = 0.125 \times 12.7 \times 9^2 = 129\ \text{kNm}$

$M_s = 0.070 w l^2 < M_h$

b) load on one span



$M_h = 0.063 w l^2 = 0.063 \times 12.7 \times 9^2 = 65\ \text{kNm}$

$M_s < 0.125 w l^2 \quad \therefore$ not critical

Moment capacity for 305 x 165 x 40 UB Grade 43

check the section for class 1 : Flange $\frac{b}{T} \leq 8.5$

Web $\frac{d}{t} \leq 64$

For 305 x 165 x 40 : $\frac{b}{T} = 8.09 < 8.5$ AND $\frac{d}{t} = 43.6 < 64$ — OK

$M_p = 624.5 \times 275 \times 10^{-3} = 172\ \text{kNm} > 129\ \text{kNm}$ — OK

Remarks

<u>SLS</u>

Load after construction $= 2.45 \times 3 = 7.35\ \text{kN/m}$

$\delta = \frac{w l^4}{185 EI} = \frac{7.35 \times 9 \times 9^3 \times 10^{3 \times 3}}{185 \times 205 \times 8523 \times 10^4} = 14.9\ \text{mm}$

$\therefore\ \delta = \frac{\text{span}}{603}$ ✓ — OK

Max. +ve moment $M_s = 0.07 w l^2$

$M_s = 0.07 \times 7.35 \times 9^2 = 42\ \text{kNm}$

steel stress $= \frac{M_s}{Z} = \frac{42 \times 10^6}{561.2 \times 10^3} = 75\ \text{N/mm}^2$ ✓ — OK

<u>Composite Stage</u>

<u>ULS</u>

4.4.1(a) — Effective breadth from 4.6.

4.4.1(b) — Concrete in ribs neglected.

4.4.1(d) — Concrete in tension neglected.

4.4.1(e) — Reinforcement in compression neglected.

4.4.1(f) — Mesh neglected.

5.2.2 — Consider simplified method :

$\frac{\text{Unfactored Imposed Load}}{\text{" Dead "}} = \frac{6.00}{3.00} = 2 < 2.5$ ✓ — OK

other conditions for simplified method are also ok.

$W = (1.6 \times 6.0 + 1.4 \times 3.0) \times 3 = 41.4\ \text{kN/m}$

Pt 3, Table 3 — $M_s = 0.79 \times 41.4 \times 9^2 / 8 = 331\ \text{kNm}$

$M_h = 0.50 \times 41.4 \times 9^2 / 8 = 210\ \text{kNm}$

Remarks

Moment capacity of composite section

a) Internal support

table 5 — For 19 x 100 mm stud in Grade 30 concrete

$Q_k = 100$ kN

5.4.4.2 — $Q_{k,hog} = 100 \times 0.6 = 60$ kN

5.4.7.2 — $k = 0.85 \left(\frac{b_r}{D_p}\right)\left(\frac{h}{D_p} - 1\right)$

$h = 95$ mm , $D_p = 46$ mm

$b_r = 105 + \frac{120 - 67}{2} = 131.5$ mm

$k = 0.85 \left(\frac{131.5}{46}\right)\left(\frac{95}{46} - 1\right) \not> 1$

$k = 1$

Total shear resistance $= 7 \times 60 = 420$ kN

$\therefore R_r = 420$ kN

For full shear connection $\frac{d}{t} = 43.6 > 38\,\varepsilon$ — $\varepsilon = 1$

Neutral Axis in top flange

B.2.4 — $M_c = R_n \frac{D}{2} + R_r D_r - \frac{(R_n - R_r)^2}{R_f} \frac{T}{4}$

$R_s = 275 \times 5150 \times 10^{-3} = 1416$ kN

$R_v = d\,t\,p_y = 265.7 \times 6.1 \times 275 \times 10^{-3} = 502$ kN

$R_o = 38 \times 1 \times 6.1^2 \times 275 \times 10^{-3} = 389$ kN

$R_n = R_s - R_v + R_o = 1360$ kN

$R_f = 165.1 \times 10.2 \times 275 \times 10^{-3} = 463$ kN

$M_c = 1360 \frac{303.8}{2} + 420 \times 90 - \frac{(1360 - 420)^2}{463} \frac{10.2}{4}$

$= 240$ kNm > 210 kNm — OK

Remarks

Moment Capacity of composite section contd.

b) Span

$b_e = 0.8 \times 9000 / 8 = 900$ mm < 1500 ✓

$B_e = 2 \times 900 = 1800$ mm

8.2.1 — $R_c = 0.45 \times 30 \times 1800 (120 - 46) \times 10^{-3} = 1798$ kN

$R_s = 5150 \times 275 \times 10^{-3} = 1416$ kN

($R_s < R_c$ $\therefore$ P.N.A. is not in concrete)

$R_f = 165.1 \times 10.2 \times 275 \times 10^{-3} = 463$ kN

$R_w = R_s - 2R_f = 1416 - 2 \times 463 = 490$ kN

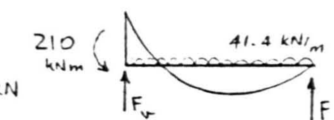
($R_c > R_w$ $\therefore$ P.N.A. in steel flange)

$M_c = R_s \frac{D}{2} + R_c \frac{(D_s + D_p)}{2} - \frac{(R_s - R_c)^2}{R_f} \cdot \frac{T}{4}$

$M_c = 1416 \frac{303.8}{2} + 1798 \frac{(120 + 46)}{2} - \frac{(1416 - 1798)^2}{463} \times \frac{10.2}{4}$

$M_c = 363$ kNm > 331 kNm — OK

Shear Capacity



$F_v = 41.4 \times \frac{9}{2} + \frac{210}{9} = 210$ kN

Assume section at support is steel only.

Pt. 1 4.2.3 — $P_v = 0.6\,p_y\,D\,t = 0.6 \times 275 \times 303.8 \times 6.1 \times 10^{-3} = 306$ kN

Pt. 3 5.3.4 — $F_v > 0.5\,P_v$

Remarks

$\therefore M_{cv} = M_c - (M_c - M_f)\left(2\frac{F_v}{P_v} - 1\right)^2$

$S_r = S - \left(\frac{1}{6}tD^2\right) \times 1.15 = 624.5 - 107.9 = 516.6$ cm³

$M_f = 516.6 \times 10^3 \times 275 \times 10^{-6} = 142$ kNm

$M_{cv} = 240 - (240 - 142)\left(2 \times \frac{210}{306} - 1\right)^2$

$= 226$ kNm > 210 kNm ✓ — OK

5.6 **Transverse reinforcement**

5.6.2 Total longitudinal shear force per unit length $v = \frac{NQ_p}{s}$

$N = 1$, $s = 225$ mm , $Q_p = 60$ kN

$v = \frac{60}{225} \times 10^3 = 267$ N/mm

5.6.3 Shear resistance $v_r = 0.7 A_{sv} f_y + 0.03 \eta A_{cv} f_{cu} + v_p$

$\leq 0.8 \eta A_{cv} \sqrt{f_{cu}} + v_p$

$A_{cv} = 97 \times 1 = 97$ mm², $A_{sv} = 0.142$ mm²/mm

$\eta = 1.0$ normal weight concrete

$v_p = t_p \cdot p_{yp} = 0.9 \times 280 = 252$ N/mm

$v_r = 0.7 \times 0.142 \times 460 + 0.03 \times 1 \times 97 \times 30 + 252$

$= 46 + 87 + 252 = 385$ N/mm

$0.8 \eta A_{cv} \sqrt{f_{cu}} + v_p = 0.8 \times 1 \times 97\sqrt{30} + 252 = 677 > 385$ ✓

$v = 267 < 385$ ✓ — OK

$\therefore$ No need for additional transverse reinforcement

Remarks

5.2.5 **Stability of bottom flange**

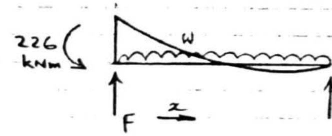
Loaded span to be checked with factored dead load only.

$M_h = M_{cv} = 226$ kNm

Design dead load $= 1.4 \times 3.00 \times 3 = 12.6$ kN/m

$F = \left(\frac{12.6 \times 9^2}{2} + 226\right)/9$

$= 82$ kN



For zero moment $F.x = 226 + \frac{wx^2}{2}$

$6.3x^2 - 82x + 226 = 0$

$x = 5.00$ m

bottom flange is in compression over 5 m.

SCI 060 p.21 $L_{cr} = 3.74\, I_y^{0.25} \left(\frac{D}{t}\right)^{0.75}$

$= 3.74 \times (763 \times 10^4)^{0.25} \left(\frac{303.8}{6.1}\right)^{0.75}$

$= 3.69$ m < 5.00 m

$M_{3.69} = 226 + 12.6\frac{(3.69)^2}{2} - 82 \times 3.69 = 9.2$ kNm

Pt.1 Table 16 $\beta = \frac{9.2}{226} = 4.07 \times 10^{-2}$

$M_o = \frac{12.6 \times 3.69^2}{8} = 21.5$ kNm

$\gamma = \frac{M}{M_o} = -\frac{226}{21.5} = -10.5$

For $\beta = 0.04$ and $\gamma = -10.5$, $n = 0.671$

SCI 078 p.47 $\lambda_{LT} = 3.0\, n \left(\frac{d}{t}\right)^{0.75} = 3.0 \times 0.671 \left(\frac{265.6}{6.1}\right)^{0.75} = 34 < 35$ — OK

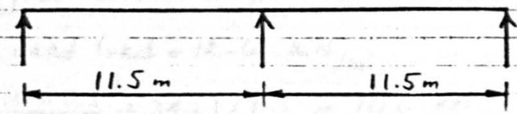
$\therefore$ No lateral restraint needed.

But $\frac{d}{t} = 44 > 38$ (Conservative)

SCI 078 p.48 However $44 < 45$ (new proposal) ✓ — OK

B-4

		Remarks



All materials, secondary spans and loadings are the same as for 305x165x40 UB.

Construction stage

Moment capacity of steel beam ($w = 12.7\ kN/m$)

Elastic section modulus $= 949 \times 10^3\ mm^3$

$M_e = 949 \times 275 \times 10^{-3} = 261\ kNm$

worse case (load on both span):

$M_h = 0.125\, w l^2 = 0.125 \times 12.7 \times 11.5^2 = 210\ kNm$ — OK.

SLS: $\delta = \dfrac{wl^4}{185 EI} = \dfrac{7.35 \times 11.5^4 \times 10^9}{185 \times 205 \times 21345 \times 10^4} = 15.9\ mm$

$\therefore \delta = \dfrac{span}{723}$ ✓ — OK.

Composite stage

$w = 41.4\ kN/m$

$M_s = 0.79 \times 41.4 \times 11.5^2/8 = 541\ kNm$

$M_h = 0.50 \times 41.4 \times 11.5^2/8 = 342\ kNm$

sagging moment resistance:

$M_{c,sag} = 579\ kNm > 541\ kNm$ ✓ — O.K.

Hogging moment resistance:

$R_r = 905 \times 460 = 416\ kN$

$R_v = d\,t\,p_y = 407.7 \times 7.6 \times 275 \times 10^{-3} = 852\ kN$

		Remarks

$R_s = A\,p_y = 6650 \times 275 \times 10^{-3} = 1829\ kN$

$R_f = B\,T\,p_y = 152.4 \times 10.9 \times 275 \times 10^{-3} = 457\ kN$

$R_w = R_s - 2R_f = 915\ kN$

$\varepsilon = \sqrt{\dfrac{275}{p_y}} = 1$

$R_o = 38\,\varepsilon\,t^2 p_y = 38 \times 1 \times 7.6^2 \times 275 \times 10^{-3} = 604\ kN$

BS5950 Case 5 — PNA in web

$\dfrac{d}{t} = \dfrac{407.7}{7.6} = 53.6 > 38$

$\dfrac{d}{t} > \dfrac{76}{1 + \frac{416}{852}} = 51.1$

$\therefore$ Web not compact

$M_c = M_s + R_r\left(\dfrac{D}{2} + D_r\right) - \dfrac{R_r^2 + (R_v + R_r)(R_v + R_r - 2R_o)}{R_v}\,\dfrac{d}{4}$

$M_s = 1094 \times 275 \times 10^{-3} = 301\ kNm$

$M_c = 301 + 416\left(\dfrac{449.8}{2} + 90\right) \times 10^{-3}$

$\quad - \dfrac{416^2 + (852 + 416)(852 + 416 - 2 \times 604)}{852}\,\dfrac{407.7}{4} \times 10^{-3}$

$M_c = 402\ kNm > 342\ kNm$ ✓ — OK

B-5

Shear capacity:

$F_v = 41.4 \times \dfrac{11.5}{2} + 342/11.5 = 268\ kN$

$P_v = 0.6\,p_y\,D\,t = 0.6 \times 275 \times 449.8 \times 7.6 \times 10^{-3}$

$= 564\ kN$

$F_v < 0.5\ P_v$

Remarks

stability of bottom flange

Design dead load = 12.6 kN/m

$F = \left(\frac{12.6 \times 11.5^2}{2} + 342\right)/11.5 = 102$ kN

$F x = 342 + \frac{w x^2}{2}$

$6.3x^2 - 102x + 342 = 0$

$x = 4.74$ m

$L_{cr} = 3.74\, I_y^{0.25} \left(\frac{D}{t}\right)^{0.75}$

$= 3.74 \times (645 \times 10^4)^{0.25} \left(\frac{449.8}{7.6}\right)^{0.75}$

$= 4.02$ m < 4.74 m

$M_{4.02} = 342 + 12.6 \frac{(4.02)^2}{2} - 102 \times 4.02 = 33.8$ kNm

$\beta = \frac{33.8}{342} = 0.0987$

$M_o = \frac{12.6 \times 4.02^2}{8} = 25$ kNm

$\gamma = \frac{M}{M_o} = -\frac{342}{25} = -13.7$

For $\beta = 0.1$ and $\gamma = -14$, $n = 0.693$

$\lambda_{LT} = 3.0\, n \left(\frac{d}{t}\right)^{0.75} = 3 \times 0.693 \left(\frac{407.7}{7.6}\right)^{0.75} = 41 > 35$

∴ Tortional restraint required.

Appendix C

Material Properties and Cross Sectional Dimensions

C-1-Notation

$f_{ys,w}$	yield strength of steel section, web
$f_{ys,f}$	yield strength of steel section, flange
$f_{us,w}$	ultimate strength of steel section, web
$f_{us,f}$	ultimate strength of steel section, flange
E_w	modulus of elasticity, web
E_f	modulus of elasticity, flange
f_{yr}	yield strength of reinforcement
f_{ur}	ultimate strength of reinforcement
E	modulus of elasticity
f_{ye}	yield strength of end plate
f_{ue}	ultimate strength of end plate
f_{yb}	yield strength of bolt
f_{ub}	ultimate strength of bolt
D	depth of steel section
B	breadth of flange of steel section
t	thickness of web
T	thickness of flange
T_t	thickness of top flange
T_b	thickness of bottom flange
B_N	breadth of flange, north side
B_S	breadth of flange, south side

C-2-Material Properties

C-2-1-Steel Sections

C-2-1(a)-Steel Beam

TEST No.	$f_{ys,w}$ N/mm^2	$f_{ys,f}$ N/mm^2	$f_{us,w}$ N/mm^2	$f_{us,f}$ N/mm^2	E_w kN/mm^2	E_f kN/mm^2
1	293	271	459	465	206	206
2	297	273	463	464	206	207
3	295	272	461	464	206	207
4	307	273	465	476	203	202
5	310	272	460	470	199	201
6	308	273	463	473	201	202
7	302	272	462	467	203	204
8	302	272	462	467	203	204
9	302	272	462	467	203	204
10	330	310	492	476	203	196
11	302	272	462	467	203	204

C-2-1(b)-Steel Column

The following values were measured and used for all tests:

$f_{ys,w}=296\ N/mm^2$

$f_{ys,f}=284\ N/mm^2$

$f_{us,w}=508\ N/mm^2$

$f_{us,f}=484\ N/mm^2$

$E_w=207\ kN/mm^2$

$E_f=205\ kN/mm^2$

C-2-2-Slab

C-2-2(a)-Reinforcement

The following values were measured and used for all tests:

Bars: $f_{yr}=486\ N/mm^2$
$f_{ur}=557\ N/mm^2$
$E=200\ kN/mm^2$
$Elongation=17\%$

Mesh: $f_{yr}=668\ N/mm^2$
$f_{ur}=712\ N/mm^2$
$Elongation=2\%$

C-2-2(b)-Concrete

TEST No.	$f_{cu,7days}$ N/mm^2	$f_{cu,28days}$ N/mm^2	AGE AT TEST day	$f_{cu,test}$ N/mm^2	$f_{t,test}$ N/mm^2
1	33.2	44.1	22	41.3	3.43
2	32.0	42.5	29	42.7	3.19
3	37.6	49.0	22	46.8	3.31
4	31.9	41.3	21	40.3	2.80
5	34.0	37.3	45	44.1	3.47
6	33.6	44.4	28	44.4	3.32
7	34.9	42.5	20	44.2	3.44
8	30.5	43.1	22	41.7	2.70
10	40.9	51.5	26	49.8	4.06

C-2-3-End plate

The end plate of Tests 1-7 were from one piece of plate with measured values of $f_{ye}=308N/mm^2$ and $f_{ue}=503N/mm^2$. The end plate of Tests 8-11 were from another piece of plate with measured values of $f_{ye}=309N/mm^2$ and $f_{ue}=500N/mm^2$.

Therefore the lesser values were used:

$f_{ye}=308\ N/mm^2$
$f_{ue}=500\ N/mm^2$

C-2-4-Bolts

All bolts were Grade 8.8 with the following properties taken from EC3:

$f_{yb}=640N/mm^2$
$f_{ub}=800N/mm^2$

C-3-Cross Sectional Dimensions

C-3-1-Major Axis Tests

C-3-1(a)-Beams

TEST No.	NORTH BEAM					SOUTH BEAM				
	D	B	t	T_t	T_b	D	B	t	T_t	T_b
1	306.2	165.5	6.35	10.20	10.09	305.6	165.5	6.31	9.89	10.14
2	305.6	165.5	6.33	10.17	9.95	305.7	165.3	6.29	10.13	9.91
3	305.7	165.5	6.38	10.15	10.27	305.5	165.5	6.41	9.95	10.13
4	305.4	165.6	6.38	9.98	10.24	306.0	165.3	6.37	10.17	9.95
7	305.8	165.1	6.40	10.00	10.10	305.7	165.0	6.40	10.15	9.95
9	303.6	164.6	6.27	9.96	9.90	305.5	164.9	6.40	10.07	10.15
10	450.9	153.8	7.93	10.32	9.95	450.9	154.0	8.00	10.31	10.27

C-3-1(b)-Columns

TEST No.	D	B_N	B_S	t	T_N	T_S
1	209.0	204.0	205.0	8.08	12.53	12.28
2	209.0	209.0	205.0	8.06	12.48	12.34
3	208.4	205.0	203.9	7.99	12.33	12.35
4	210.0	203.9	205.1	7.79	12.44	12.39
7	208.8	203.4	204.6	7.90	12.00	12.20
9	210.2	203.7	204.7	7.98	12.40	12.25
10	208.8	204.0	205.0	8.02	12.39	12.37

C-3-2-Minor Axis Tests

C-3-2(a)-Beams

TEST No.	NORTH BEAM					SOUTH BEAM				
	D	B	t	T_t	T_b	D	B	t	T_t	T_b
5	305.4	165.4	6.42	10.21	10.20	305.4	165.5	6.40	10.17	10.20
6	305.1	165.5	6.35	10.28	9.98	305.0	165.4	6.38	10.10	10.19
8	304.3	164.9	6.20	9.85	9.90	304.9	165.1	6.20	9.80	9.95
11	305.1	164.8	6.34	9.91	10.18	305.5	165.0	6.36	9.96	10.13

C-3-2(b)-Columns

TEST No.	D	B	t	T
5	209.2	204.7	7.55	12.35
6	208.6	204.2	7.80	12.25
8	209.3	204.2	7.96	12.20
11	209.8	204.4	7.54	12.28

C-3-3-End Plate Thicknesses

In the following table, the first row gives the number of test, the second and the third rows give the end plate thicknesses for north and south sides respectively.

1	2	3	4	5	6	7	8	9	10	11
15.22	15.26	15.23	15.35	15.25	15.28	15.20	15.20	15.25	15.20	15.19
15.26	15.50	15.20	15.28	15.20	15.30	15.17	15.20	15.19	15.18	15.20

C-3-4-Reinforcement

The diameter of reinforcing bars was measured with two different methods and an average of 12.09 mm was found but 12 mm was used. The measured diameter of mesh bars was 5.86 mm.

C-3-5-Profiled Steel Sheeting

The geometric dimensions of PMF CF46 were taken from the manufacturer's guide. The measured thickness of decking was 0.9 mm.

C-3-6-Stud Connectors

TRW Nelson studs were used in Tests 1-6, and Crompton & Parkinson studs in Tests 7,8 and 10. The diameter of studs was measured as 19.0 mm. The heights of studs after welding were as follows:

TEST No.	1	2	3	4	5	6	7	8	10
MINIMUM	95.00	95.50	93.10	93.05	95.00	95.25	96.25	97.45	96.60
MAXIMUM	98.00	98.00	95.15	95.65	97.50	97.60	98.15	99.20	98.90
AVERAGE	96.50	96.75	94.12	94.35	96.25	96.42	97.20	98.32	97.75

Appendix D

Resistance Values of Connections Based on the Actual Measured Geometric and Mechanical Properties (North and South Connections)

PARAMETERS
(Units N and mm)

1) DIMENSIONS:

DB	depth of beam
BWT	beam web thickness
BFBT	beam flange breadth, top
BFTT	beam flange thickness, top
BFBB	beam flange breadth, bottom
BFTB	beam flange thickness, bottom
RRB	root radius of beam section
BL	beam length
DC	depth of column
CWT	column web thickness
CFB	column flange breadth
CFT	column flange thickness
RRC	root radius of column section
EPT	end plate thickness
EPB	end plate breadth
BPH	bolt pitch, horizontal
BME	distance from the center of top bolt row to top of top flange or if extended end plate, to top of end plate
BPV	bolt pitch, vertical (used only in extended end plate)
WAF	specified weld throat of beam flange
WAW	specified weld throat of beam web
ABOLT	area of bolt
DBOLT	diameter of bolt
AR	area of reinforcement
DR	depth of reinforcement(to top of the top flange)
AM	area of mesh
DM	depth of mesh(to top of the top flange)
DS	depth of slab
DP	depth of profiled steel sheeting

2) STRENGTHS:

FYBF	yield strength of beam flange
FYBW	yield strength of beam web
FYCF	yield strength of column flange
FYCW	yield strength of column web
FYEP	yield strength of end plate
FUB	ultimate strength of bolt
FYR	yield strength of reinforcement
FYM	yield strength of mesh
FCU	cube strength of concrete

TEST 1(N)

1) DIMENSIONS:

BEAM:	DB=306.2	BWT= 6.35	BFBT=165.5	BFTT=10.20
	BFBB=165.5	BFTB=10.09	RRB= 8.90	BL= 9000.0
COLUMN:	DC=209.0	CWT= 8.08	CFB=204.0	CFT=12.53
	RRC=10.20			
END PLATE:	EPT=15.22	EPB=200.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR= 905.0	DR= 90.0	AM=162.0	DM= 87.0
PROFILED STEEL SHEETING:		DS=120.0	DP=46.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=271.0	FYBW=293.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR=486.0	FYM=668.0
CUBE STRENGTH OF CONCRETE:	FCU=41.3	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION= 295.62 kN

RESISTANCE OF COLUMN FLANGE IN TENSION= 258.26 kN

RESISTANCE OF COLUMN WEB IN TENSION= 462.84 kN

MINIMUM RESISTANCE IN TENSION ZONE= 258.26 kN

RESISTANCE MOMENT OF STEEL CONNECTION= 64.86 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS ONLY)= 219.46 kNm
RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS AND MESH)= 243.49 kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5241
I= 85975792 Z= 563320)

RESISTANCE MOMENT OF STEEL SECTION= 172.73 kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)= 371.30 kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS ONLY)= 248.45 kNm
REBARS AND MESH)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS AND MESH)= 258.96 kNm

TEST 1(S)

1) DIMENSIONS:

BEAM:	DB=305.6	BWT= 6.31	BFBT=165.5	BFTT= 9.89
	BFBB=165.5	BFTB=10.14	RRB= 8.90	BL= 9000.0
COLUMN:	DC=209.0	CWT= 8.08	CFB=205.0	CFT=12.28
	RRC=10.20			
END PLATE:	EPT=15.26	EPB=200.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR= 905.0	DR= 90.0	AM=162.0	DM= 87.0
PROFILED STEEL SHEETING:		DS=120.0	DP=46.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=271.0	FYBW=293.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR=486.0	FYM=668.0
CUBE STRENGTH OF CONCRETE:	FCU=41.3	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION= 296.14 kN

RESISTANCE OF COLUMN FLANGE IN TENSION= 255.80 kN

RESISTANCE OF COLUMN WEB IN TENSION= 462.84 kN

MINIMUM RESISTANCE IN TENSION ZONE= 255.80 kN

RESISTANCE MOMENT OF STEEL CONNECTION= 64.08 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS ONLY)= 218.94 kNm
RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS AND MESH)= 243.07 kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5184
I= 84674080 Z= 550209)

RESISTANCE MOMENT OF STEEL SECTION= 170.45 kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)= 367.19 kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS ONLY)= 245.74 kNm
REBARS AND MESH)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS AND MESH)= 256.20 kNm

TEST 2(N)

1) DIMENSIONS:

BEAM:	DB=305.6	BWT= 6.33	BFBT=165.5	BFTT=10.17
	BFBB=165.5	BFTB= 9.95	RRB= 8.90	BL= 9000.0
COLUMN:	DC=209.0	CWT= 8.06	CFB=209.0	CFT=12.48
	RRC=10.20			
END PLATE:	EPT=15.26	EPB=200.0	BPH= 86.0	BME= 52.0
	BPV= 90.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR= 905.0	DR= 90.0	AM=162.0	DM= 87.0
PROFILED STEEL SHEETING:		DS=120.0	DP=46.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=273.0	FYBW=297.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR=486.0	FYM=668.0
CUBE STRENGTH OF CONCRETE:	FCU=42.7	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION (EXTENDED PART)=	224.17	kN
(BELOW THE TENSION FLANGE)=	296.14	kN
RESISTANCE OF COLUMN FLANGE IN TENSION=	416.49	kN
RESISTANCE OF COLUMN WEB IN TENSION=	692.11	kN
RESISTANCE MOMENT OF STEEL CONNECTION=	122.29	kNm
RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS ONLY)=	248.07	kNm
RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS AND MESH)=	262.95	kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5204
I= 85006440 Z= 559838)

RESISTANCE MOMENT OF STEEL SECTION=	172.62	kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)= 372.67 kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS ONLY)= 247.99 kNm
REBARS AND MESH)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS AND MESH)= 258.51 kNm

TEST 2(S)

1) DIMENSIONS:

BEAM:	DB=305.7	BWT= 6.29	BFBT=165.3	BFTT=10.13
	BFBB=165.3	BFTB= 9.91	RRB= 8.90	BL= 9000.0
COLUMN:	DC=209.0	CWT= 8.06	CFB=205.0	CFT=12.34
	RRC=10.20			
END PLATE:	EPT=15.50	EPB=200.0	BPH= 86.0	BME= 52.0
	BPV= 90.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR= 905.0	DR= 90.0	AM=162.0	DM= 87.0
PROFILED STEEL SHEETING:		DS=120.0	DP=46.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=273.0	FYBW=297.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR=486.0	FYM=668.0
CUBE STRENGTH OF CONCRETE:	FCU=42.7	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION (EXTENDED PART)=	231.13	kN
(BELOW THE TENSION FLANGE)=	299.32	kN
RESISTANCE OF COLUMN FLANGE IN TENSION=	403.69	kN
RESISTANCE OF COLUMN WEB IN TENSION=	686.14	kN
RESISTANCE MOMENT OF STEEL CONNECTION=	118.58	kNm
RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS ONLY)=	246.44	kNm
RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS AND MESH)=	261.77	kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5177
I= 84643520 Z= 557281)

RESISTANCE MOMENT OF STEEL SECTION=	171.80	kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)= 370.92 kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS ONLY)= 246.57 kNm
REBARS AND MESH)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS AND MESH)= 257.08 kNm

TEST 3(N)

1) DIMENSIONS:

BEAM:	DB=305.7	BWT= 6.38	BFBT=165.5	BFTT=10.15
	BFBB=165.5	BFTB=10.27	RRB= 8.90	BL= 9000.0
COLUMN:	DC=208.4	CWT= 7.99	CFB=205.0	CFT=12.33
	RRC=10.20			
END PLATE:	EPT=15.23	EPB=200.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR= 452.0	DR= 90.0	AM=162.0	DM= 87.0
PROFILED STEEL SHEETING:		DS=120.0	DP=46.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=272.0	FYBW=295.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR=486.0	FYM=668.0
CUBE STRENGTH OF CONCRETE:	FCU=46.8	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION= 295.75 kN

RESISTANCE OF COLUMN FLANGE IN TENSION= 256.28 kN

RESISTANCE OF COLUMN WEB IN TENSION= 458.34 kN

MINIMUM RESISTANCE IN TENSION ZONE= 256.28 kN

RESISTANCE MOMENT OF STEEL CONNECTION= 64.22 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION
(REBARS ONLY)= 149.89 kNm
RESISTANCE MOMENT OF COMPOSITE CONNECTION
(REBARS AND MESH)= 187.38 kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5267
I= 86141648 Z= 561672)

RESISTANCE MOMENT OF STEEL SECTION= 174.10 kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)= 377.42 kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
PLASTIC NEUTRAL AXIS IN WEB , WEB COMPACT
HOGGING MOMENT RESISTANCE(REBARS ONLY)= 221.04 kNm
REBARS AND MESH)
PLASTIC NEUTRAL AXIS IN WEB , WEB COMPACT
HOGGING MOMENT RESISTANCE(REBARS AND MESH)= 239.12 kNm

TEST 3(S)

1) DIMENSIONS:

BEAM:	DB=305.5	BWT= 6.41	BFBT=165.5	BFTT= 9.95
	BFBB=165.5	BFTB=10.13	RRB= 8.90	BL= 9000.0
COLUMN:	DC=208.4	CWT= 7.99	CFB=203.9	CFT=12.35
	RRC=10.20			
END PLATE:	EPT=15.20	EPB=200.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR= 452.0	DR= 90.0	AM=162.0	DM= 87.0
PROFILED STEEL SHEETING:		DS=120.0	DP=46.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=272.0	FYBW=295.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR=486.0	FYM=668.0
CUBE STRENGTH OF CONCRETE:	FCU=46.8	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION= 295.36 kN

RESISTANCE OF COLUMN FLANGE IN TENSION= 256.48 kN

RESISTANCE OF COLUMN WEB IN TENSION= 458.34 kN

MINIMUM RESISTANCE IN TENSION ZONE= 256.48 kN

RESISTANCE MOMENT OF STEEL CONNECTION= 64.23 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION
(REBARS ONLY)= 149.79 kNm
RESISTANCE MOMENT OF COMPOSITE CONNECTION
(REBARS AND MESH)= 186.92 kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5220
I= 84971488 Z= 553446)

RESISTANCE MOMENT OF STEEL SECTION= 172.04 kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)= 374.28 kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
PLASTIC NEUTRAL AXIS IN WEB , WEB COMPACT
HOGGING MOMENT RESISTANCE(REBARS ONLY)= 218.98 kNm
REBARS AND MESH)
PLASTIC NEUTRAL AXIS IN WEB , WEB COMPACT
HOGGING MOMENT RESISTANCE(REBARS AND MESH)= 237.10 kNm

TEST 4(N)

1) DIMENSIONS:

BEAM:	DB=305.4	BWT= 6.38	BFBT=165.5	BFTT= 9.98
	BFBB=165.6	BFTB=10.24	RRB= 8.90	BL= 9000.0
COLUMN:	DC=210.0	CWT= 7.79	CFB=203.9	CFT=12.44
	RRC=10.20			
END PLATE:	EPT=15.35	EPB=200.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR=1357.0	DR= 90.0	AM=162.0	DM= 87.0
PROFILED STEEL	SHEETING:	DS=120.0	DP=46.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=273.0	FYBW=307.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR=486.0	FYM=668.0
CUBE STRENGTH OF CONCRETE:	FCU=40.3	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION=	297.33	kN
RESISTANCE OF COLUMN FLANGE IN TENSION=	257.36	kN
RESISTANCE OF COLUMN WEB IN TENSION=	448.32	kN
MINIMUM RESISTANCE IN TENSION ZONE=	257.36	kN
RESISTANCE MOMENT OF STEEL CONNECTION=	64.41	kNm
RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS ONLY)=	266.88	kNm
RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS AND MESH)=	280.17	kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5234
I= 85322568 Z= 554568)

RESISTANCE MOMENT OF STEEL SECTION= 174.73 kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)= 377.40 kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS ONLY)= 272.93 kNm
REBARS AND MESH)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS AND MESH)= 283.17 kNm

TEST 4(S)

1) DIMENSIONS:

BEAM:	DB=305.4	BWT= 6.38	BFBT=165.5	BFTT= 9.98
	BFBB=165.6	BFTB=10.24	RRB= 8.90	BL= 9000.0
COLUMN:	DC=210.0	CWT= 7.79	CFB=203.9	CFT=12.44
	RRC=10.20			
END PLATE:	EPT=15.35	EPB=200.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR=1357.0	DR= 90.0	AM=162.0	DM= 87.0
PROFILED STEEL	SHEETING:	DS=120.0	DP=46.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=273.0	FYBW=307.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR=486.0	FYM=668.0
CUBE STRENGTH OF CONCRETE:	FCU=40.3	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION=	297.33	kN
RESISTANCE OF COLUMN FLANGE IN TENSION=	257.36	kN
RESISTANCE OF COLUMN WEB IN TENSION=	448.32	kN
MINIMUM RESISTANCE IN TENSION ZONE=	257.36	kN
RESISTANCE MOMENT OF STEEL CONNECTION=	64.41	kNm
RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS ONLY)=	266.88	kNm
RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS AND MESH)=	280.17	kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5234
I= 85322568 Z= 554568)

RESISTANCE MOMENT OF STEEL SECTION= 174.73 kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)= 377.40 kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS ONLY)= 272.93 kNm
REBARS AND MESH)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS AND MESH)= 283.17 kNm

TEST 5(N)

1) DIMENSIONS:

BEAM:	DB=305.4	BWT= 6.42	BFBT=165.4	BFTT=10.21
	BFBB=165.4	BFTB=10.20	RRB= 8.90	BL= 9000.0
COLUMN:	DC=209.2	CWT= 7.55	CFB=204.7	CFT=12.35
	RRC=10.20			
END PLATE:	EPT=15.25	EPB=166.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR= 905.0	DR= 90.0	AM=162.0	DM= 87.0
PROFILED STEEL SHEETING:		DS=120.0	DP=46.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=272.0	FYBW=310.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR=486.0	FYM=668.0
CUBE STRENGTH OF CONCRETE:	FCU=44.1	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION= 296.01 kN

RESISTANCE OF TOP ROW BOLTS IN TENSION= 352.80 kN

MINIMUM RESISTANCE IN TENSION ZONE= 296.01 kN

RESISTANCE MOMENT OF STEEL CONNECTION= 74.09 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS ONLY)= 225.07 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS AND MESH)= 248.43 kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5273
I= 85954944 Z= 563058)

RESISTANCE MOMENT OF STEEL SECTION= 175.94 kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)= 382.90 kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
PLASTIC NEUTRAL AXIS IN WEB , WEB NOT COMPACT
HOGGING MOMENT RESISTANCE(REBARS ONLY)= 251.43 kNm
REBARS AND MESH)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS AND MESH)= 263.32 kNm

TEST 5(S)

1) DIMENSIONS:

BEAM:	DB=305.4	BWT= 6.40	BFBT=165.5	BFTT=10.17
	BFBB=165.5	BFTB=10.20	RRB= 8.90	BL= 9000.0
COLUMN:	DC=209.2	CWT= 7.55	CFB=204.7	CFT=12.35
	RRC=10.20			
END PLATE:	EPT=15.20	EPB=166.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR= 905.0	DR= 90.0	AM=162.0	DM= 87.0
PROFILED STEEL SHEETING:		DS=120.0	DP=46.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=272.0	FYBW=310.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR=486.0	FYM=668.0
CUBE STRENGTH OF CONCRETE:	FCU=44.1	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION= 295.36 kN

RESISTANCE OF TOP ROW BOLTS IN TENSION= 352.80 kN

MINIMUM RESISTANCE IN TENSION ZONE= 295.36 kN

RESISTANCE MOMENT OF STEEL CONNECTION= 73.93 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS ONLY)= 224.99 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS AND MESH)= 248.34 kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5263
I= 85831504 Z= 561617)

RESISTANCE MOMENT OF STEEL SECTION= 175.65 kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)= 382.21 kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
PLASTIC NEUTRAL AXIS IN WEB , WEB NOT COMPACT
HOGGING MOMENT RESISTANCE(REBARS ONLY)= 250.55 kNm
REBARS AND MESH)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS AND MESH)= 262.74 kNm

TEST 6 (N)

1) DIMENSIONS:

BEAM:	DB=305.1	BWT= 6.35	BFBT=165.5	BFTT=10.28
	BFBB=165.5	BFTB= 9.98	RRB= 8.90	BL= 9000.0
COLUMN:	DC=208.6	CWT= 7.80	CFB=204.2	CFT=12.25
	RRC=10.20			
END PLATE:	EPT=15.28	EPB=166.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR= 0.0	DR= 0.0	AM=162.0	DM= 92.0
PROFILED STEEL SHEETING:		DS=120.0	DP=46.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=273.0	FYBW=308.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR=486.0	FYM=668.0
CUBE STRENGTH OF CONCRETE:	FCU=44.4	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION= 296.41 kN

RESISTANCE OF TOP ROW BOLTS IN TENSION= 352.80 kN

MINIMUM RESISTANCE IN TENSION ZONE= 296.41 kN

RESISTANCE MOMENT OF STEEL CONNECTION= 74.13 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS ONLY)= 74.13 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS AND MESH)= 116.57 kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5229 I= 85182480 Z= 563183)

RESISTANCE MOMENT OF STEEL SECTION= 174.65 kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)= 379.77 kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
PLASTIC NEUTRAL AXIS IN WEB , WEB COMPACT
HOGGING MOMENT RESISTANCE(REBARS ONLY)= 174.65 kNm
REBARS AND MESH)
PLASTIC NEUTRAL AXIS IN WEB , WEB COMPACT
HOGGING MOMENT RESISTANCE(REBARS AND MESH)= 199.62 kNm

TEST 6 (S)

1) DIMENSIONS:

BEAM:	DB=305.0	BWT= 6.38	BFBT=165.4	BFTT=10.10
	BFBB=165.4	BFTB=10.19	RRB= 8.90	BL= 9000.0
COLUMN:	DC=208.6	CWT= 7.80	CFB=204.2	CFT=12.25
	RRC=10.20			
END PLATE:	EPT=15.30	EPB=166.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR= 0.0	DR= 0.0	AM=162.0	DM= 92.0
PROFILED STEEL SHEETING:		DS=120.0	DP=46.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=273.0	FYBW=308.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR=486.0	FYM=668.0
CUBE STRENGTH OF CONCRETE:	FCU=44.4	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION= 296.67 kN

RESISTANCE OF TOP ROW BOLTS IN TENSION= 352.80 kN

MINIMUM RESISTANCE IN TENSION ZONE= 296.67 kN

RESISTANCE MOMENT OF STEEL CONNECTION= 74.14 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS ONLY)= 74.14 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION (REBARS AND MESH)= 116.55 kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5240 I= 85239456 Z= 557528)

RESISTANCE MOMENT OF STEEL SECTION= 174.89 kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)= 380.44 kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
PLASTIC NEUTRAL AXIS IN WEB , WEB COMPACT
HOGGING MOMENT RESISTANCE(REBARS ONLY)= 174.89 kNm
REBARS AND MESH)
PLASTIC NEUTRAL AXIS IN WEB , WEB COMPACT
HOGGING MOMENT RESISTANCE(REBARS AND MESH)= 199.86 kNm

TEST 7(N)

1) DIMENSIONS:

BEAM:	DB=305.8	BWT= 6.40	BFBT=165.1	BFTT=10.00
	BFBB=165.1	BFTB=10.10	RRB= 8.90	BL= 9000.0
COLUMN:	DC=208.8	CWT= 7.90	CFB=203.4	CFT=12.00
	RRC=10.20			
END PLATE:	EPT=15.20	EPB=200.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR=1357.0	DR= 90.0	AM=162.0	DM= 87.0
PROFILED STEEL SHEETING:		DS=120.0	DP=46.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=272.0	FYBW=302.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR=486.0	FYM=668.0
CUBE STRENGTH OF CONCRETE:	FCU=44.2	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION= 295.36 kN

RESISTANCE OF COLUMN FLANGE IN TENSION= 253.10 kN

RESISTANCE OF COLUMN WEB IN TENSION= 453.84 kN

MINIMUM RESISTANCE IN TENSION ZONE= 253.10 kN

RESISTANCE MOMENT OF STEEL CONNECTION= 63.46 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION
(REBARS ONLY)= 264.34 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION
(REBARS AND MESH)= 277.02 kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5214
I= 85030136 Z= 554541)

RESISTANCE MOMENT OF STEEL SECTION= 172.92 kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)= 375.91 kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS ONLY)= 271.28 kNm
REBARS AND MESH)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS AND MESH)= 281.52 kNm

TEST 7(S)

1) DIMENSIONS:

BEAM:	DB=305.7	BWT= 6.40	BFBT=165.0	BFTT=10.15
	BFBB=165.0	BFTB= 9.95	RRB= 8.90	BL= 9000.0
COLUMN:	DC=208.8	CWT= 7.90	CFB=204.6	CFT=12.20
	RRC=10.20			
END PLATE:	EPT=15.17	EPB=200.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR=1357.0	DR= 90.0	AM=162.0	DM= 87.0
PROFILED STEEL SHEETING:		DS=120.0	DP=46.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=272.0	FYBW=302.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR=486.0	FYM=668.0
CUBE STRENGTH OF CONCRETE:	FCU=44.2	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION= 294.97 kN

RESISTANCE OF COLUMN FLANGE IN TENSION= 255.02 kN

RESISTANCE OF COLUMN WEB IN TENSION= 453.84 kN

MINIMUM RESISTANCE IN TENSION ZONE= 255.02 kN

RESISTANCE MOMENT OF STEEL CONNECTION= 63.94 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION
(REBARS ONLY)= 262.64 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION
(REBARS AND MESH)= 274.83 kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5212
I= 84920800 Z= 558754)

RESISTANCE MOMENT OF STEEL SECTION= 172.75 kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)= 375.66 kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS ONLY)= 271.03 kNm
REBARS AND MESH)
WEB NOT COMPACT
PLASTIC NEUTRAL AXIS IN STEEL FLANGE
HOGGING MOMENT RESISTANCE(REBARS AND MESH)= 281.30 kNm

TEST 8(N)

1) DIMENSIONS:

BEAM:	DB=304.3	BWT= 6.20	BFBT=164.9	BFTT= 9.85
	BFBB=164.9	BFTB= 9.90	RRB= 8.90	BL= 9000.0
COLUMN:	DC=209.3	CWT= 7.96	CFB=204.2	CFT=12.20
	RRC=10.20			
END PLATE:	EPT=15.20	EPB=166.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR= 452.0	DR= 90.0	AM=162.0	DM= 87.0
PROFILED STEEL SHEETING:		DS=120.0	DP=46.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=272.0	FYBW=302.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR=486.0	FYM=668.0
CUBE STRENGTH OF CONCRETE:	FCU=41.7	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION= 295.36 kN

RESISTANCE OF TOP ROW BOLTS IN TENSION= 352.80 kN

MINIMUM RESISTANCE IN TENSION ZONE= 295.36 kN

RESISTANCE MOMENT OF STEEL CONNECTION= 73.65 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION
(REBARS ONLY)= 157.48 kNm
RESISTANCE MOMENT OF COMPOSITE CONNECTION
(REBARS AND MESH)= 191.52 kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5088
I= 82509256 Z= 541500)

RESISTANCE MOMENT OF STEEL SECTION= 168.31 kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)= 364.83 kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
PLASTIC NEUTRAL AXIS IN WEB , WEB COMPACT
HOGGING MOMENT RESISTANCE(REBARS ONLY)= 215.06 kNm
REBARS AND MESH)
PLASTIC NEUTRAL AXIS IN WEB , WEB COMPACT
HOGGING MOMENT RESISTANCE(REBARS AND MESH)= 233.03 kNm

TEST 8(S)

1) DIMENSIONS:

BEAM:	DB=304.9	BWT= 6.20	BFBT=165.1	BFTT= 9.80
	BFBB=165.1	BFTB= 9.95	RRB= 8.90	BL= 9000.0
COLUMN:	DC=209.3	CWT= 7.96	CFB=204.2	CFT=12.20
	RRC=10.20			
END PLATE:	EPT=15.20	EPB=166.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR= 452.0	DR= 90.0	AM=162.0	DM= 87.0
PROFILED STEEL SHEETING:		DS=120.0	DP=46.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=272.0	FYBW=302.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR=486.0	FYM=668.0
CUBE STRENGTH OF CONCRETE:	FCU=41.7	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION= 295.36 kN

RESISTANCE OF TOP ROW BOLTS IN TENSION= 352.80 kN

MINIMUM RESISTANCE IN TENSION ZONE= 295.36 kN

RESISTANCE MOMENT OF STEEL CONNECTION= 73.82 kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION
(REBARS ONLY)= 157.89 kNm
RESISTANCE MOMENT OF COMPOSITE CONNECTION
(REBARS AND MESH)= 192.16 kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5096
I= 82956432 Z= 541788)

RESISTANCE MOMENT OF STEEL SECTION= 168.89 kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)= 365.78 kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
PLASTIC NEUTRAL AXIS IN WEB , WEB COMPACT
HOGGING MOMENT RESISTANCE(REBARS ONLY)= 215.70 kNm
REBARS AND MESH)
PLASTIC NEUTRAL AXIS IN WEB , WEB COMPACT
HOGGING MOMENT RESISTANCE(REBARS AND MESH)= 233.70 kNm

TEST 9(N)

1) DIMENSIONS:

BEAM:	DB=303.6	BWT= 6.27	BFBT=164.6	BFTT= 9.96
	BFBB=164.6	BFTB= 9.90	RRB= 8.90	BL= 6000.0
COLUMN:	DC=210.2	CWT= 7.98	CFB=203.7	CFT=12.40
	RRC=10.20			
END PLATE:	EPT=15.25	EPB=200.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR= 0.0	DR= 0.0	AM= 0.0	DM= 0.0
PROFILED STEEL	SHEETING:	DS= 0.0	DP= 0.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=272.0	FYBW=302.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR= 0.0	FYM= 0.0
CUBE STRENGTH OF CONCRETE:	FCU= 0.0	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION=	296.01	kN
RESISTANCE OF COLUMN FLANGE IN TENSION=	256.97	kN
RESISTANCE OF COLUMN WEB IN TENSION=	457.84	kN
MINIMUM RESISTANCE IN TENSION ZONE=	256.97	kN
RESISTANCE MOMENT OF STEEL CONNECTION=	63.90	kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5116
I= 82442560 Z= 544042)

RESISTANCE MOMENT OF STEEL SECTION=	168.67	kNm

TEST 9(S)

1) DIMENSIONS:

BEAM:	DB=305.5	BWT= 6.40	BFBT=164.9	BFTT=10.07
	BFBB=164.9	BFTB=10.15	RRB= 8.90	BL= 6000.0
COLUMN:	DC=210.2	CWT= 7.98	CFB=204.7	CFT=12.25
	RRC=10.20			
END PLATE:	EPT=15.19	EPB=200.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR= 0.0	DR= 0.0	AM= 0.0	DM= 0.0
PROFILED STEEL	SHEETING:	DS= 0.0	DP= 0.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=272.0	FYBW=302.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR= 0.0	FYM= 0.0
CUBE STRENGTH OF CONCRETE:	FCU= 0.0	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION=	295.23	kN
RESISTANCE OF COLUMN FLANGE IN TENSION=	255.50	kN
RESISTANCE OF COLUMN WEB IN TENSION=	457.84	kN
MINIMUM RESISTANCE IN TENSION ZONE=	255.50	kN
RESISTANCE MOMENT OF STEEL CONNECTION=	63.98	kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5228
I= 85143632 Z= 556147)

RESISTANCE MOMENT OF STEEL SECTION=	173.27	kNm

TEST10(N)

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1) DIMENSIONS:

BEAM:           DB=450.9     BWT= 7.93    BFBT=153.8   BFTT=10.32
                BFBB=153.8   BFTB= 9.95   RRB=10.20    BL=11500.0
COLUMN:         DC=208.8     CWT= 8.02    CFB=204.0    CFT=12.39
                RRC=10.20
END PLATE:      EPT=15.20    EPB=200.0    BPH= 86.0    BME= 50.0
                BPV=  0.0
WELDS:          WAF= 7.0     WAW= 4.0
BOLTS:          ABOLT=245.0  DBOLT=20.0
REINFORCEMENT:  AR= 905.0    DR= 90.0     AM=162.0     DM= 87.0
PROFILED STEEL SHEETING:     DS=120.0     DP=46.0

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:          FYBF=310.0    FYBW=330.0
YIELD STRENGTHS OF STEEL COLUMN:        FYCF=284.0    FYCW=296.0
YIELD STRENGTH OF END PLATE:            FYEP=308.0
ULTIMATE STRENGTH OF BOLTS:             FUB=800.0
YIELD STRENGTHS OF REINFORCEMENT:       FYR=486.0     FYM=668.0
CUBE  STRENGTH OF CONCRETE:             FCU=49.8

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION=             295.36      kN

RESISTANCE OF COLUMN FLANGE IN TENSION=         256.87      kN

RESISTANCE OF COLUMN WEB IN TENSION=            459.85      kN

MINIMUM RESISTANCE IN TENSION ZONE=             256.87      kN

RESISTANCE MOMENT OF STEEL CONNECTION=          101.70      kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION
                            (REBARS ONLY)=      326.87      kNm
RESISTANCE MOMENT OF COMPOSITE CONNECTION
                        (REBARS AND MESH)=      372.57      kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 6621
                  I= 204189728     Z= 912793)

RESISTANCE MOMENT OF STEEL SECTION=             334.28      kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)=        694.56      kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
PLASTIC NEUTRAL AXIS IN WEB , WEB NOT COMPACT
HOGGING MOMENT RESISTANCE(REBARS ONLY)=         443.90      kNm
REBARS AND MESH)
PLASTIC NEUTRAL AXIS IN WEB , WEB NOT COMPACT
HOGGING MOMENT RESISTANCE(REBARS AND MESH)=     449.97      kNm
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TEST10(S)

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1) DIMENSIONS:

BEAM:           DB=450.9     BWT= 8.00    BFBT=154.0   BFTT=10.31
                BFBB=154.0   BFTB=10.27   RRB=10.20    BL=11500.0
COLUMN:         DC=208.8     CWT= 8.02    CFB=205.0    CFT=12.37
                RRC=10.20
END PLATE:      EPT=15.18    EPB=200.0    BPH= 86.0    BME= 50.0
                BPV=  0.0
WELDS:          WAF= 7.0     WAW= 4.0
BOLTS:          ABOLT=245.0  DBOLT=20.0
REINFORCEMENT:  AR= 905.0    DR= 90.0     AM=162.0     DM= 87.0
PROFILED STEEL SHEETING:     DS=120.0     DP=46.0

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:          FYBF=310.0    FYBW=330.0
YIELD STRENGTHS OF STEEL COLUMN:        FYCF=284.0    FYCW=296.0
YIELD STRENGTH OF END PLATE:            FYEP=308.0
ULTIMATE STRENGTH OF BOLTS:             FUB=800.0
YIELD STRENGTHS OF REINFORCEMENT:       FYR=486.0     FYM=668.0
CUBE  STRENGTH OF CONCRETE:             FCU=49.8

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION=             295.10      kN

RESISTANCE OF COLUMN FLANGE IN TENSION=         256.68      kN

RESISTANCE OF COLUMN WEB IN TENSION=            459.85      kN

MINIMUM RESISTANCE IN TENSION ZONE=             256.68      kN

RESISTANCE MOMENT OF STEEL CONNECTION=          101.58      kNm

RESISTANCE MOMENT OF COMPOSITE CONNECTION
                            (REBARS ONLY)=      328.12      kNm
RESISTANCE MOMENT OF COMPOSITE CONNECTION
                        (REBARS AND MESH)=      374.54      kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 6701
                  I= 206971520     Z= 918800)

RESISTANCE MOMENT OF STEEL SECTION=             338.66      kNm

RESISTANCE MOMENT OF COMPOSITE SECTION:

POSITIVE MOMENT RESISTANCE:
PLASTIC NEUTRAL AXIS IN CONCRETE FLANGE
SAGGING MOMENT RESISTANCE(MESH IGNORED)=        702.34      kNm

NEGATIVE MOMENT RESISTANCE:
REBARS ONLY)
PLASTIC NEUTRAL AXIS IN WEB , WEB NOT COMPACT
HOGGING MOMENT RESISTANCE(REBARS ONLY)=         450.91      kNm
REBARS AND MESH)
PLASTIC NEUTRAL AXIS IN WEB , WEB NOT COMPACT
HOGGING MOMENT RESISTANCE(REBARS AND MESH)=     457.31      kNm
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TEST11(N)

1) DIMENSIONS:

BEAM:	DB=305.1	BWT= 6.34	BFBT=164.8	BFTT= 9.91
	BFBB=164.8	BFTB=10.18	RRB= 8.90	BL= 6000.0
COLUMN:	DC=209.8	CWT= 7.54	CFB=204.4	CFT=12.28
	RRC=10.20			
END PLATE:	EPT=15.19	EPB=166.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR= 0.0	DR= 0.0	AM= 0.0	DM= 0.0
PROFILED STEEL	SHEETING:	DS= 0.0	DP= 0.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=272.0	FYBW=302.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR= 0.0	FYM= 0.0
CUBE STRENGTH OF CONCRETE:	FCU= 0.0	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION= 295.23 kN

RESISTANCE OF TOP ROW BOLTS IN TENSION= 352.80 kN

MINIMUM RESISTANCE IN TENSION ZONE= 295.23 kN

RESISTANCE MOMENT OF STEEL CONNECTION= 73.81 kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5185
I= 84309768 Z= 548449)

RESISTANCE MOMENT OF STEEL SECTION= 171.72 kNm

TEST11(S)

1) DIMENSIONS:

BEAM:	DB=305.5	BWT= 6.36	BFBT=165.0	BFTT= 9.96
	BFBB=165.0	BFTB=10.13	RRB= 8.90	BL= 6000.0
COLUMN:	DC=209.8	CWT= 7.54	CFB=204.4	CFT=12.28
	RRC=10.20			
END PLATE:	EPT=15.20	EPB=166.0	BPH= 86.0	BME= 50.0
	BPV= 0.0			
WELDS:	WAF= 7.0	WAW= 4.0		
BOLTS:	ABOLT=245.0	DBOLT=20.0		
REINFORCEMENT:	AR= 0.0	DR= 0.0	AM= 0.0	DM= 0.0
PROFILED STEEL	SHEETING:	DS= 0.0	DP= 0.0	

2) STRENGTHS:

YIELD STRENGTHS OF STEEL BEAM:	FYBF=272.0	FYBW=302.0
YIELD STRENGTHS OF STEEL COLUMN:	FYCF=284.0	FYCW=296.0
YIELD STRENGTH OF END PLATE:	FYEP=308.0	
ULTIMATE STRENGTH OF BOLTS:	FUB=800.0	
YIELD STRENGTHS OF REINFORCEMENT:	FYR= 0.0	FYM= 0.0
CUBE STRENGTH OF CONCRETE:	FCU= 0.0	

3) RESISTANCES:

RESISTANCE OF END PLATE IN TENSION= 295.36 kN

RESISTANCE OF TOP ROW BOLTS IN TENSION= 352.80 kN

MINIMUM RESISTANCE IN TENSION ZONE= 295.36 kN

RESISTANCE MOMENT OF STEEL CONNECTION= 73.97 kNm

(BEAM CROSS SECTIONAL PROPERTIES: A= 5198
I= 84688112 Z= 551752)

RESISTANCE MOMENT OF STEEL SECTION= 172.30 kNm

Appendix E

Experimental Curves

a) Moment-rotation curves measured by transducers (Tests 5-11)

b) Moment-rotation curves measured by manual inclinometers on the Z-shaped plate (major axis tests)

c) Moment-strain curves measured by strain gauges on reinforcement (North side of Tests 1-3)

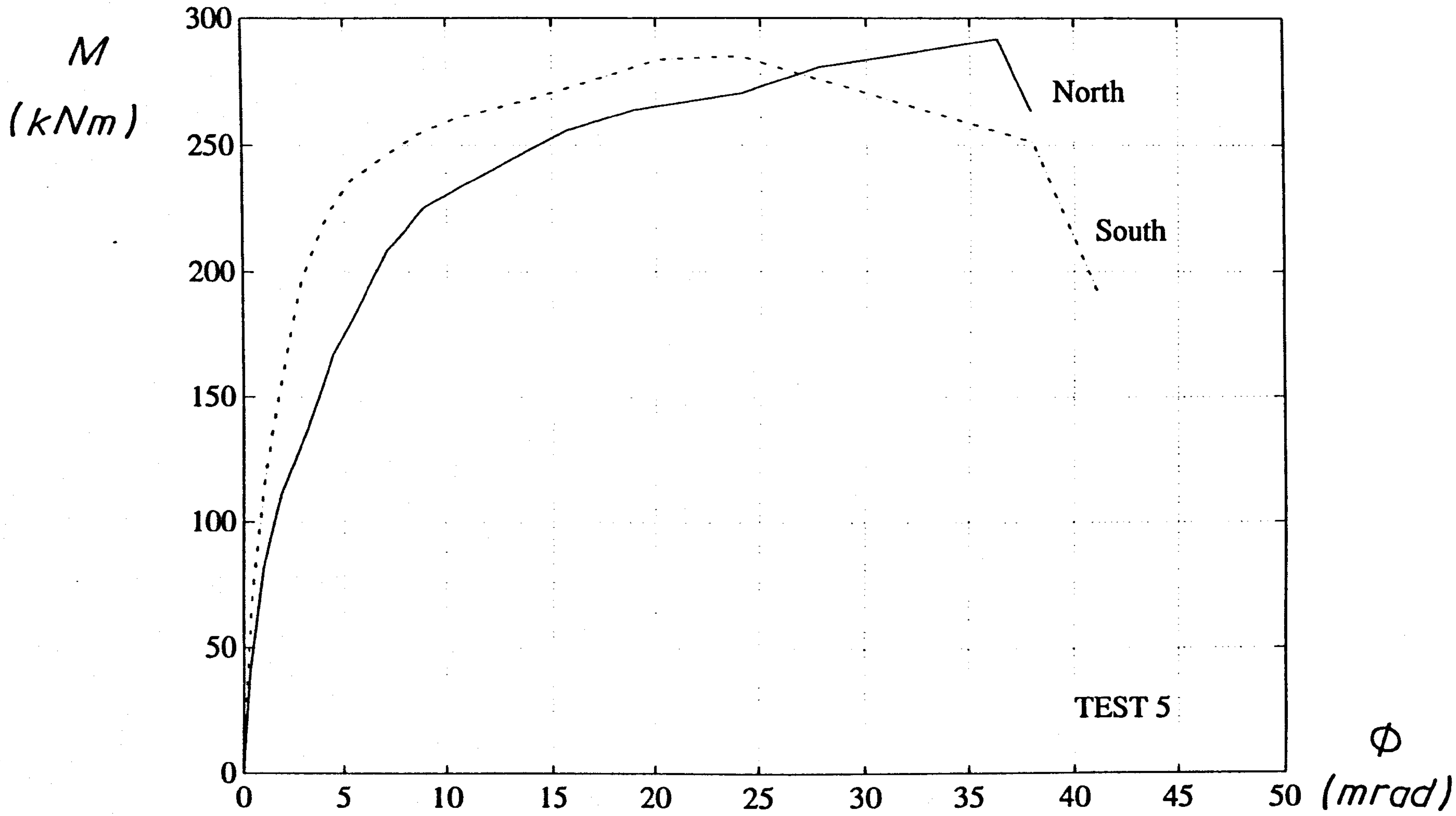


Fig. E-1, Moment-rotation curves measured by transducers in Test 5

E-1

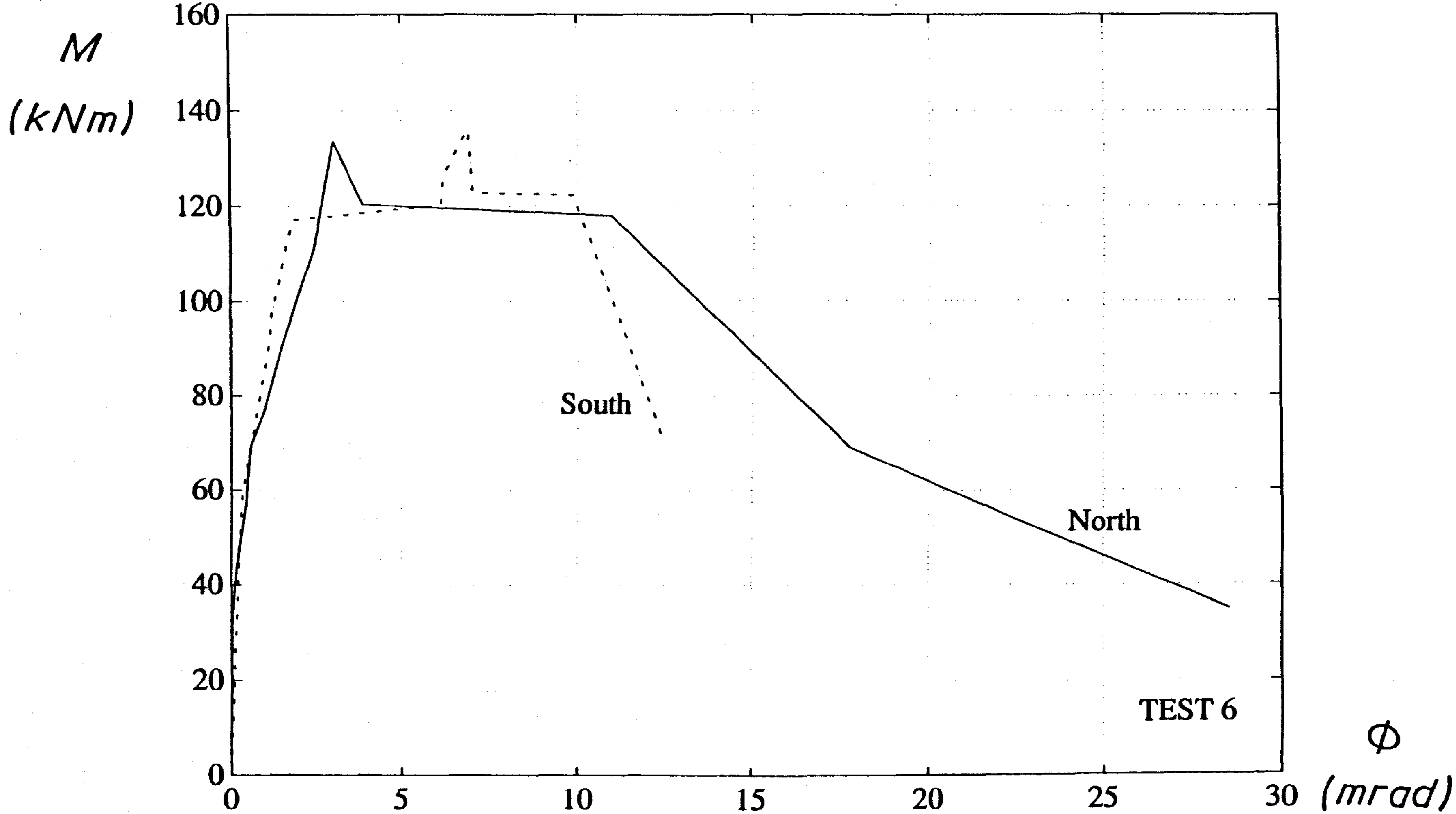


Fig. E-2, Moment-rotation curves measured by transducers in Test 6

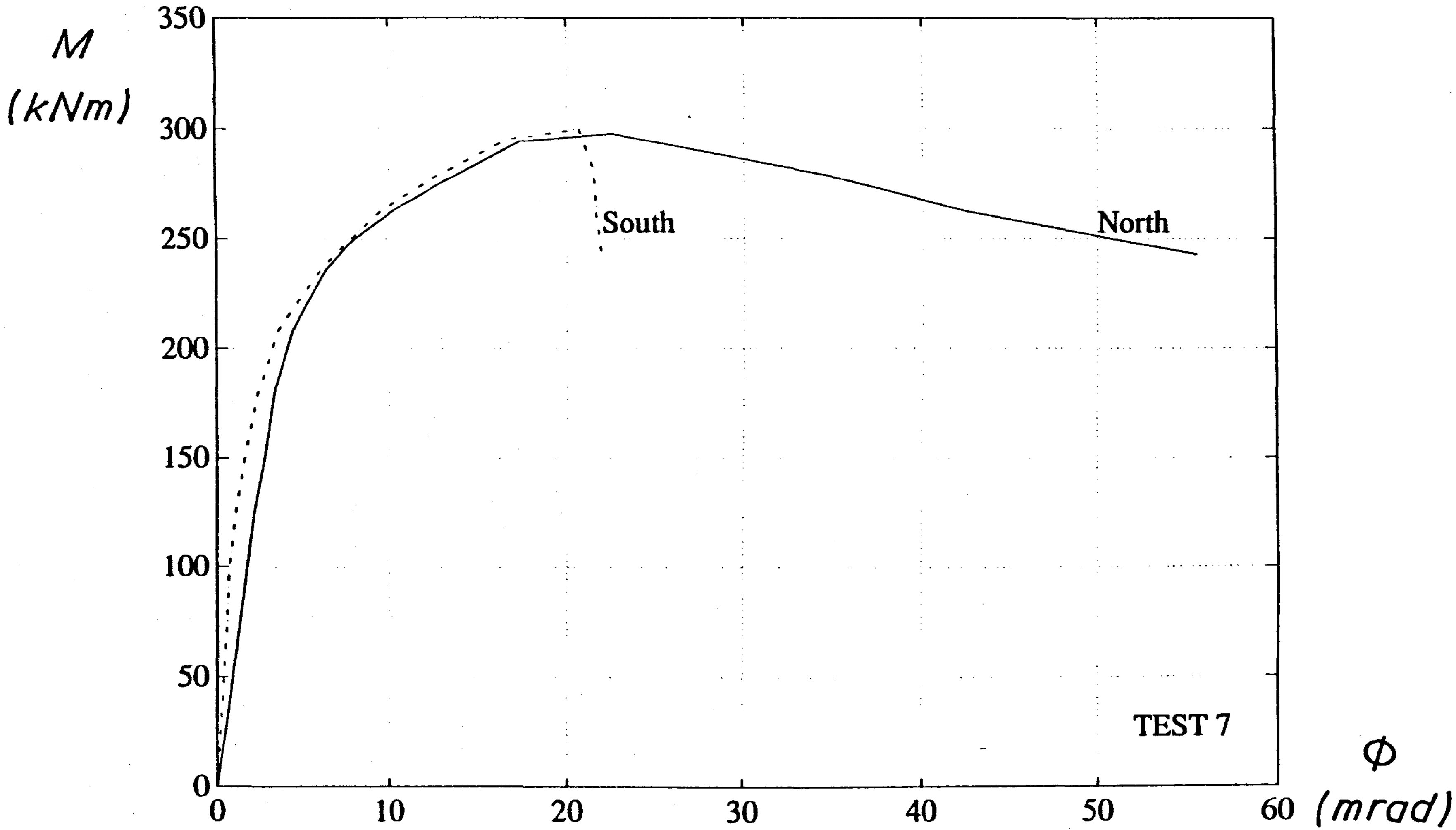


Fig. E-3, Moment-rotation curves measured by transducers in Test 7

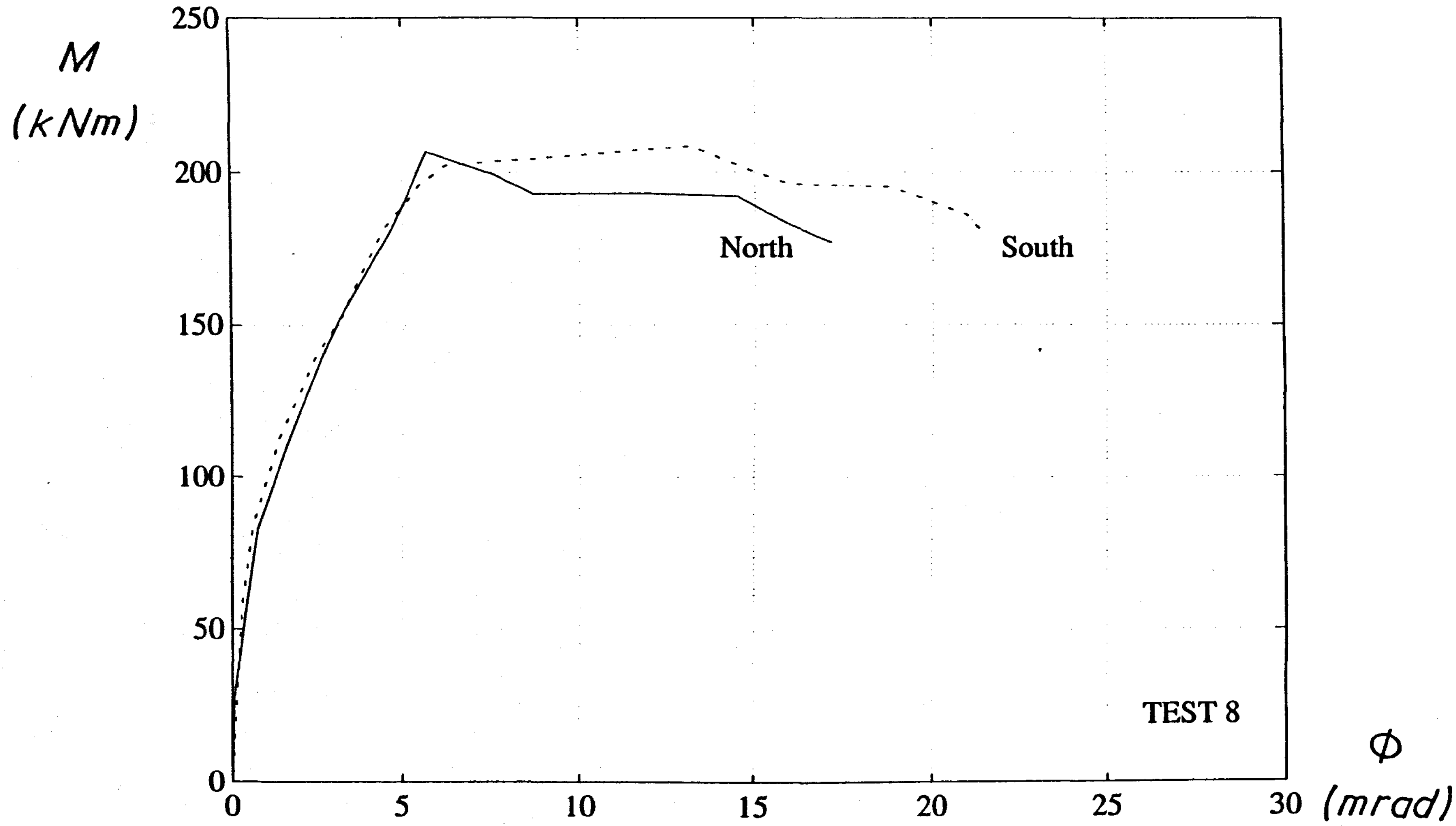


Fig. E-4, Moment-rotation curves measured by transducers in Test 8

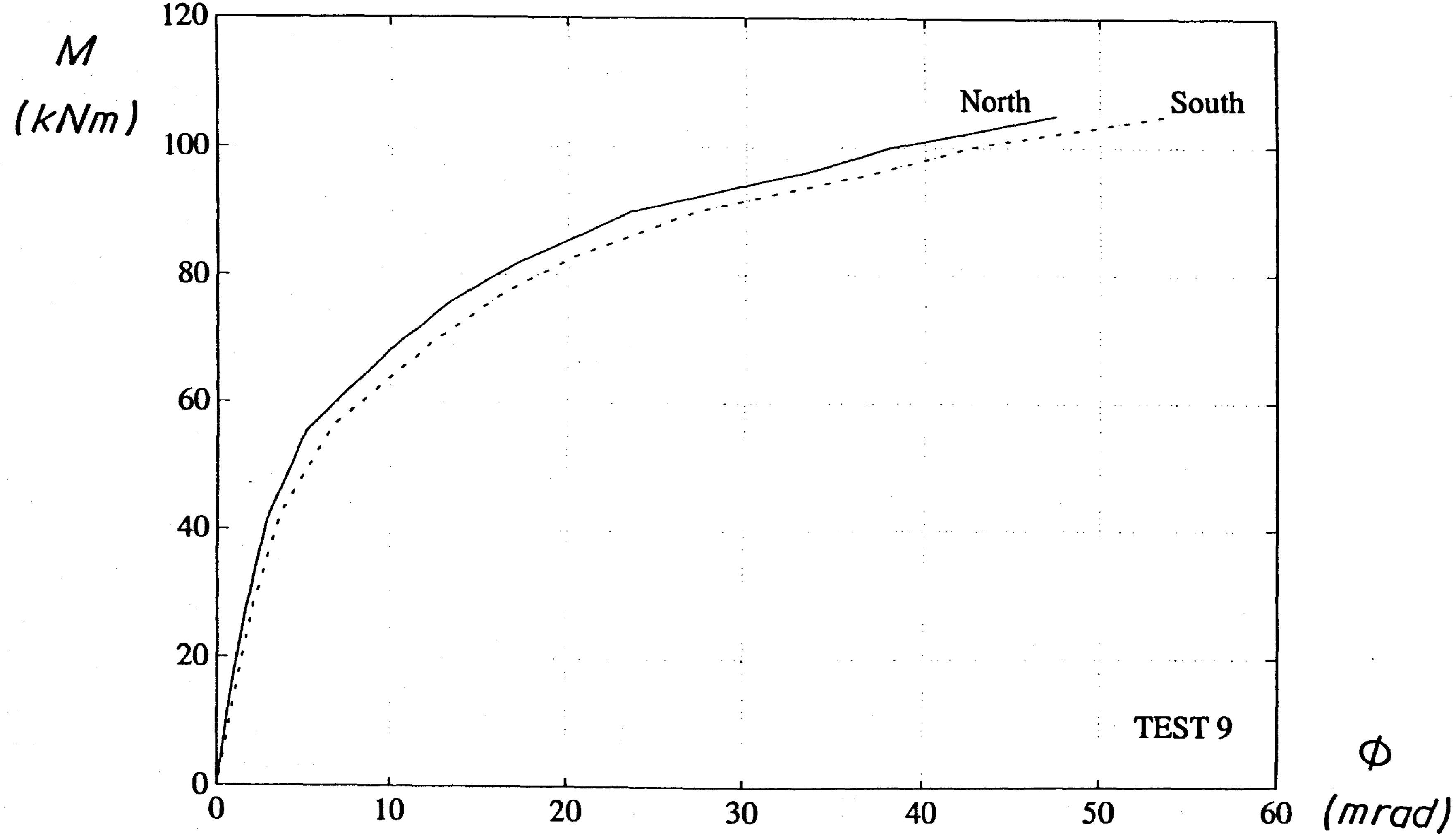


Fig. E-5, Moment-rotation curves measured by transducers in Test 9

E-5

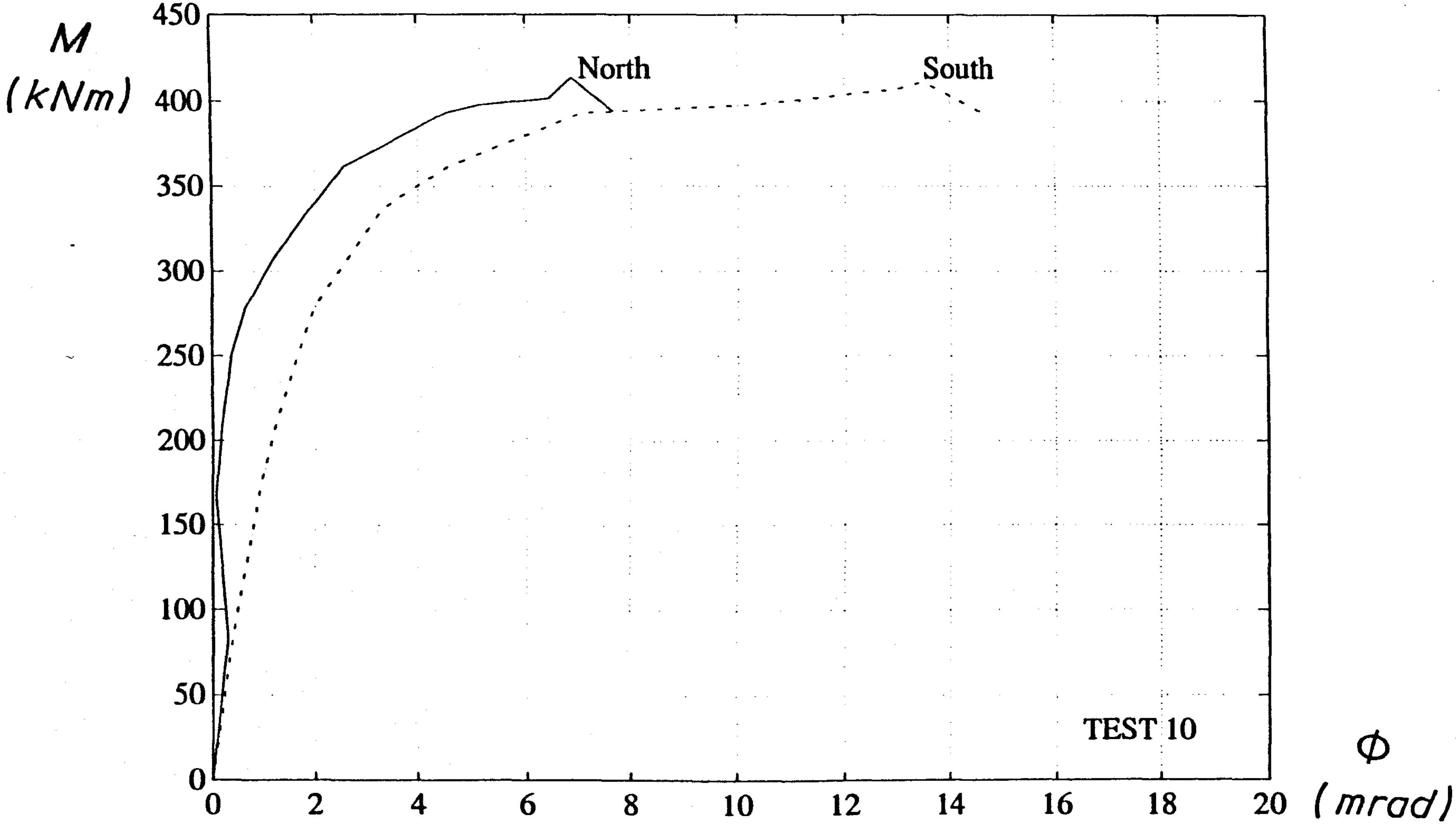


Fig. E-6, Moment-rotation curves measured by transducers in Test 10

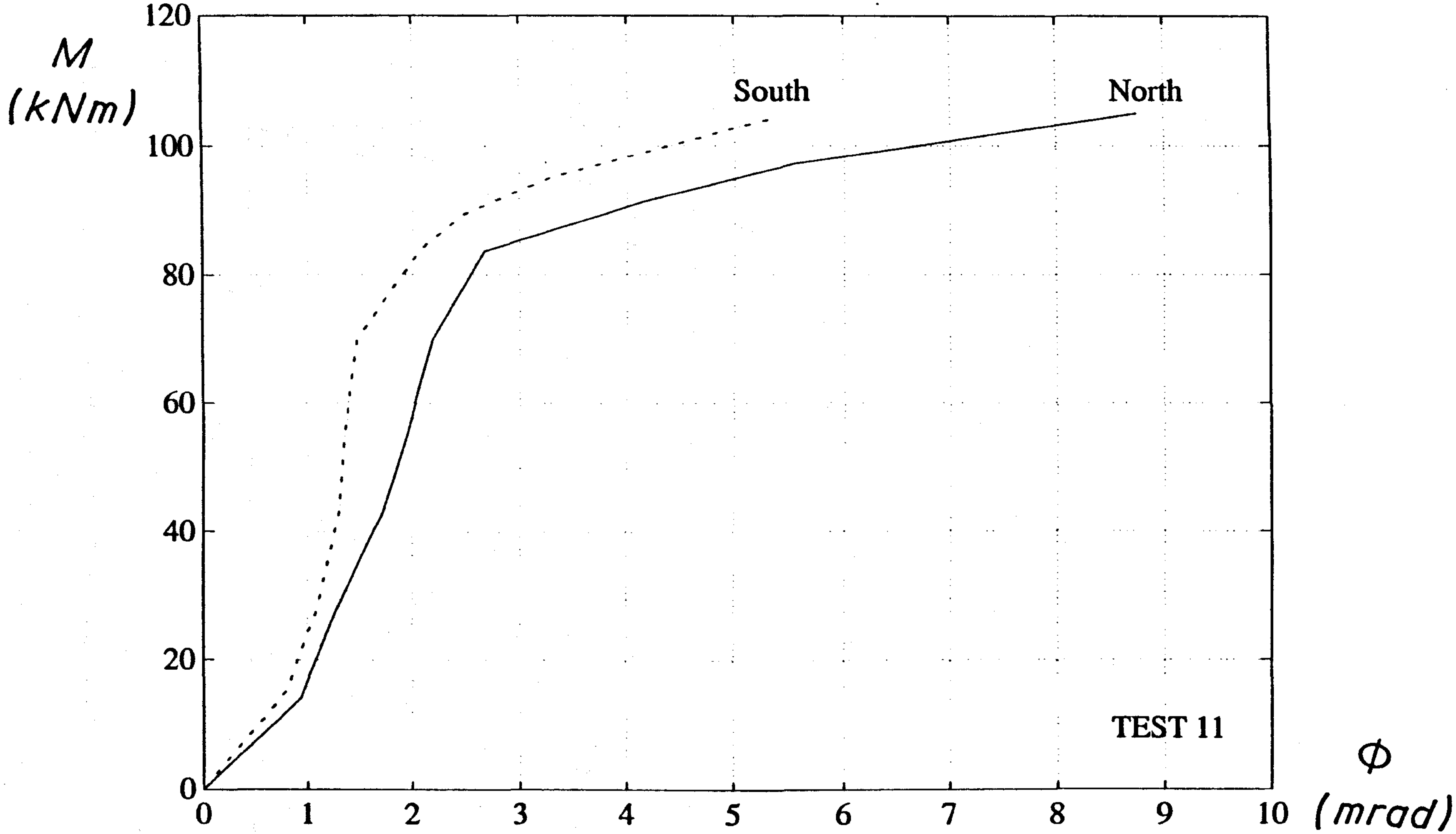


Fig. E-7, Moment-rotation curves measured by transducers in Test 11

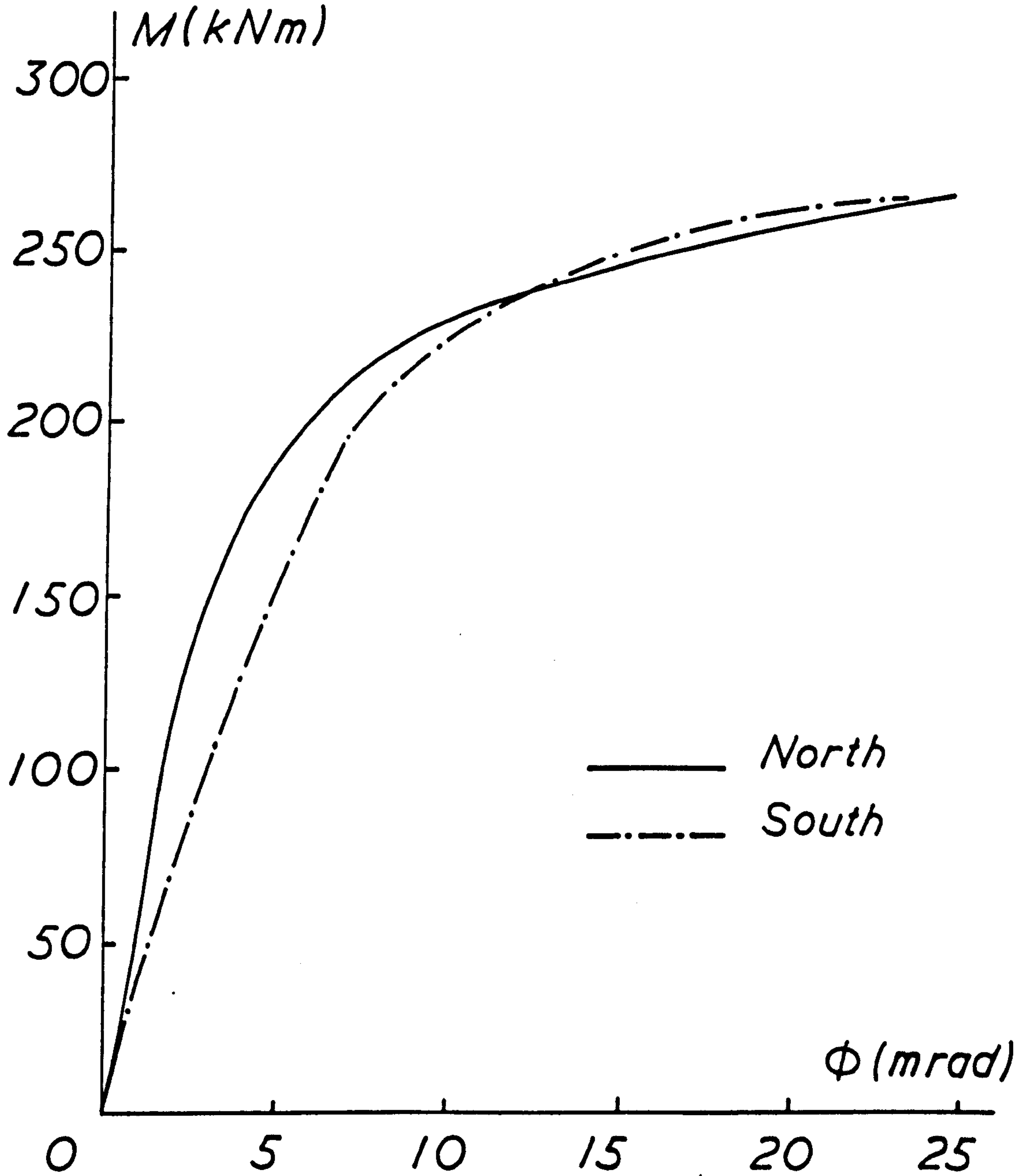


Fig. E-8, Moment-rotation curves measured by manual inclinometer in Test 1

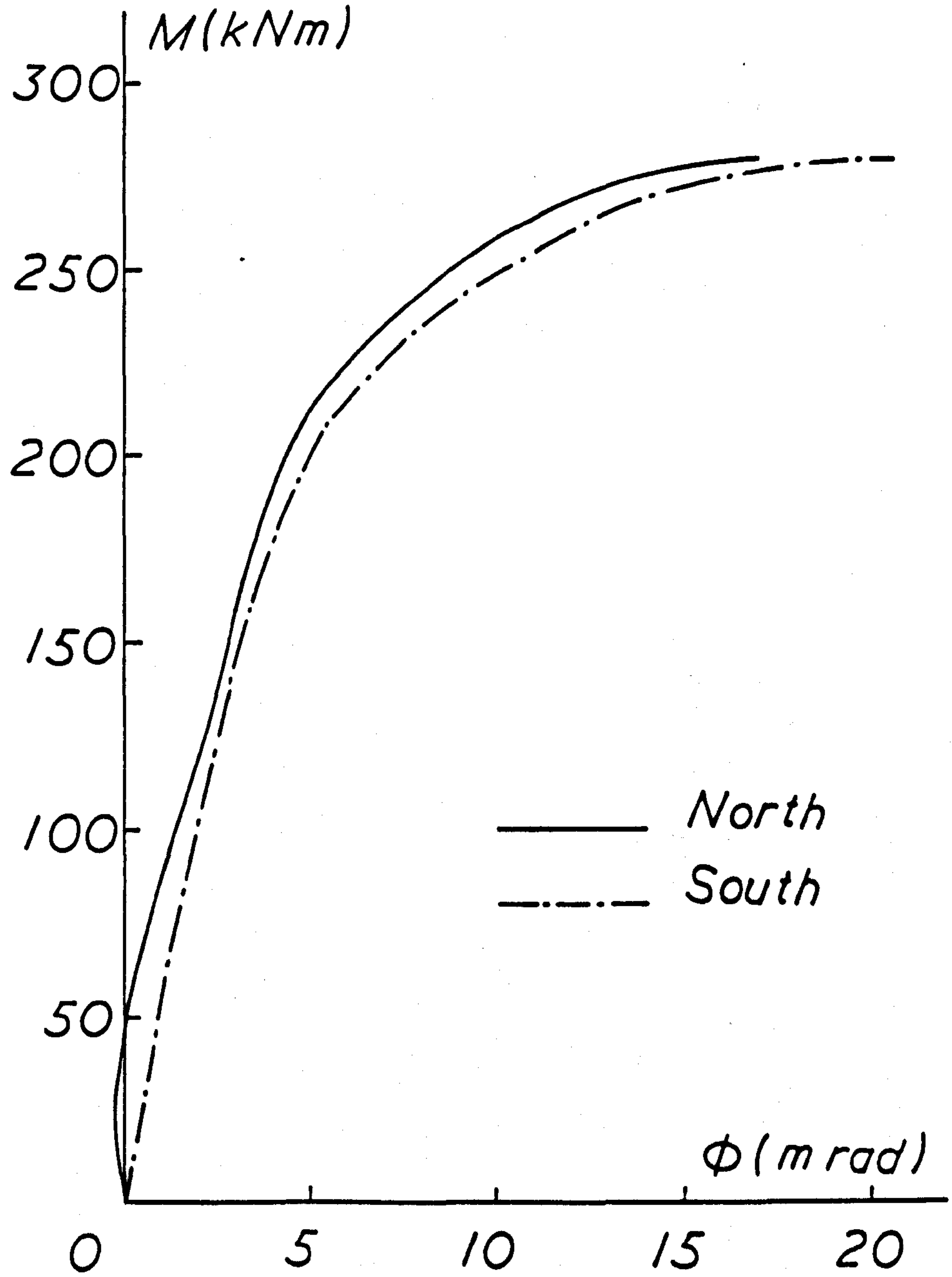


Fig. E-9, Moment-rotation curves measured by manual inclinometer in Test 2

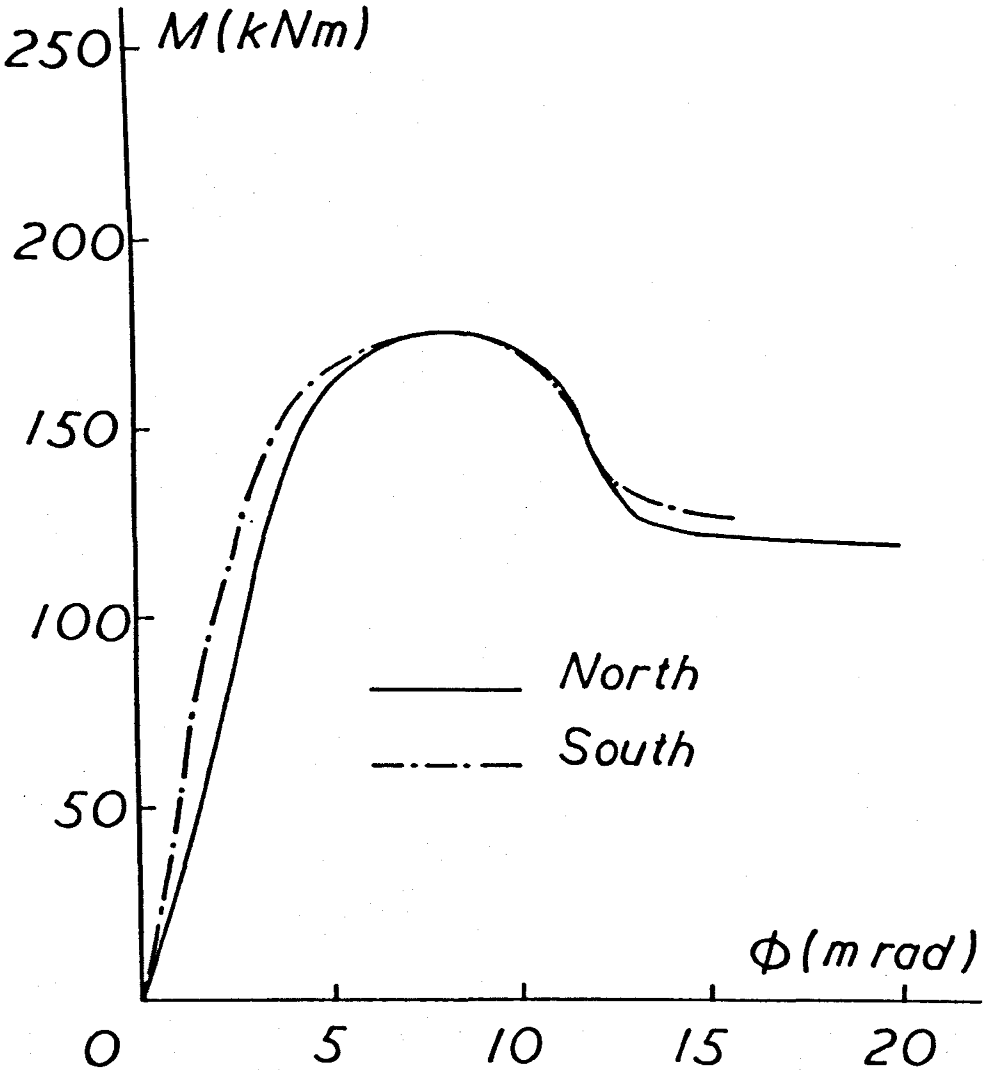


Fig. E-10, Moment-rotation curves measured by

manual inclinometer in Test 3

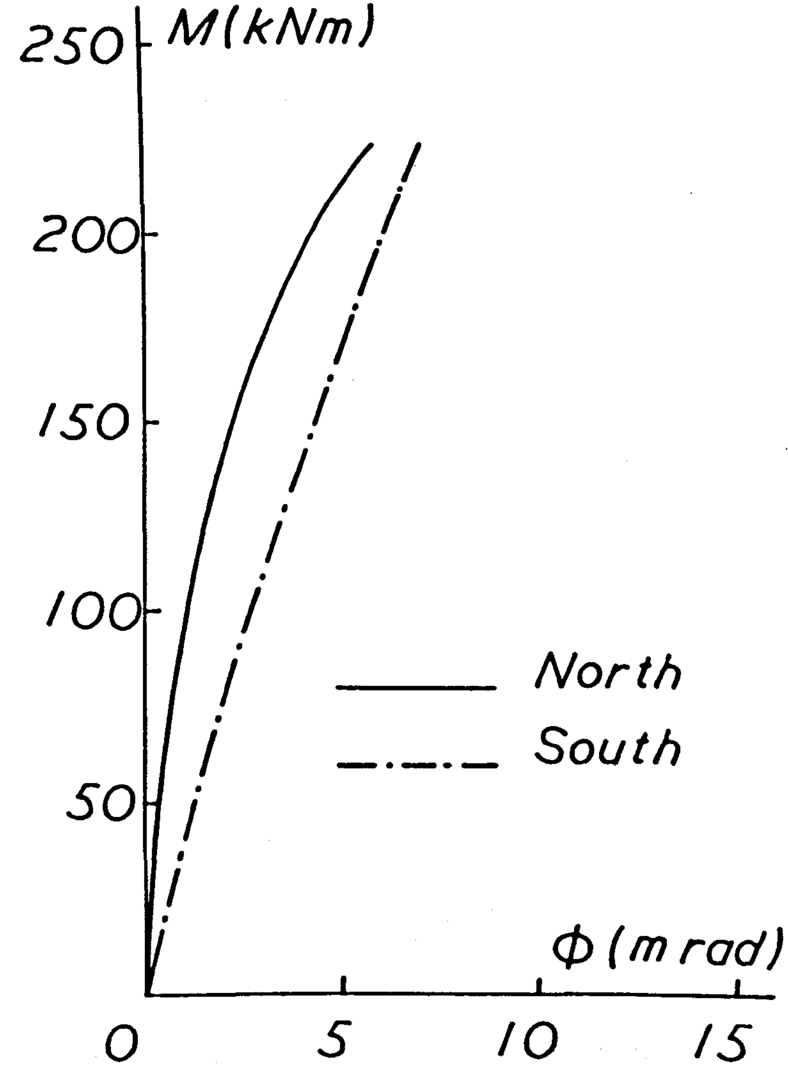


Fig. E-11, Moment-rotation curves measured by

manual inclinometer in Test 4

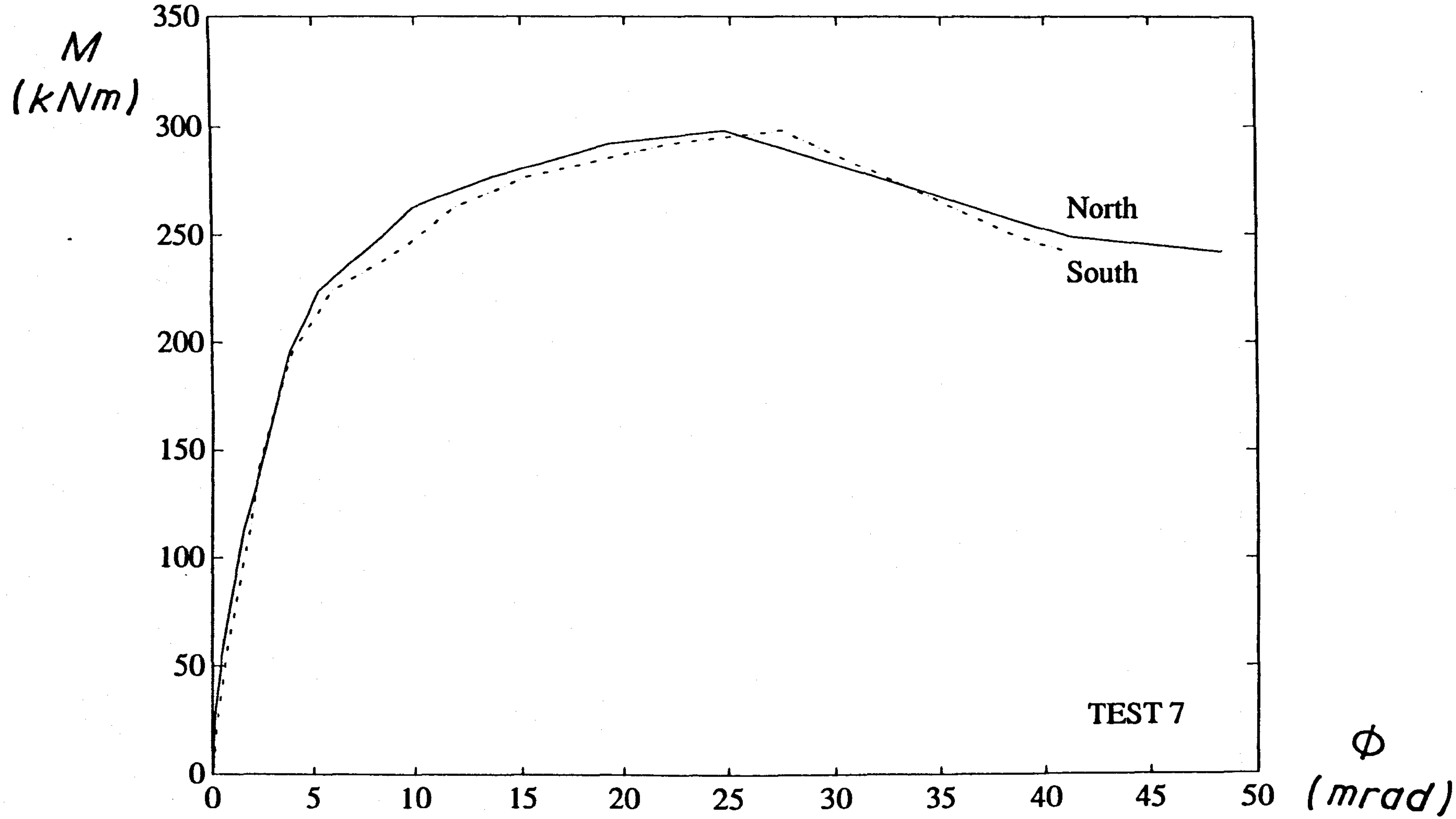


Fig. E-12, Moment-rotation curves measured by manual inclinometer in Test 7

E-11

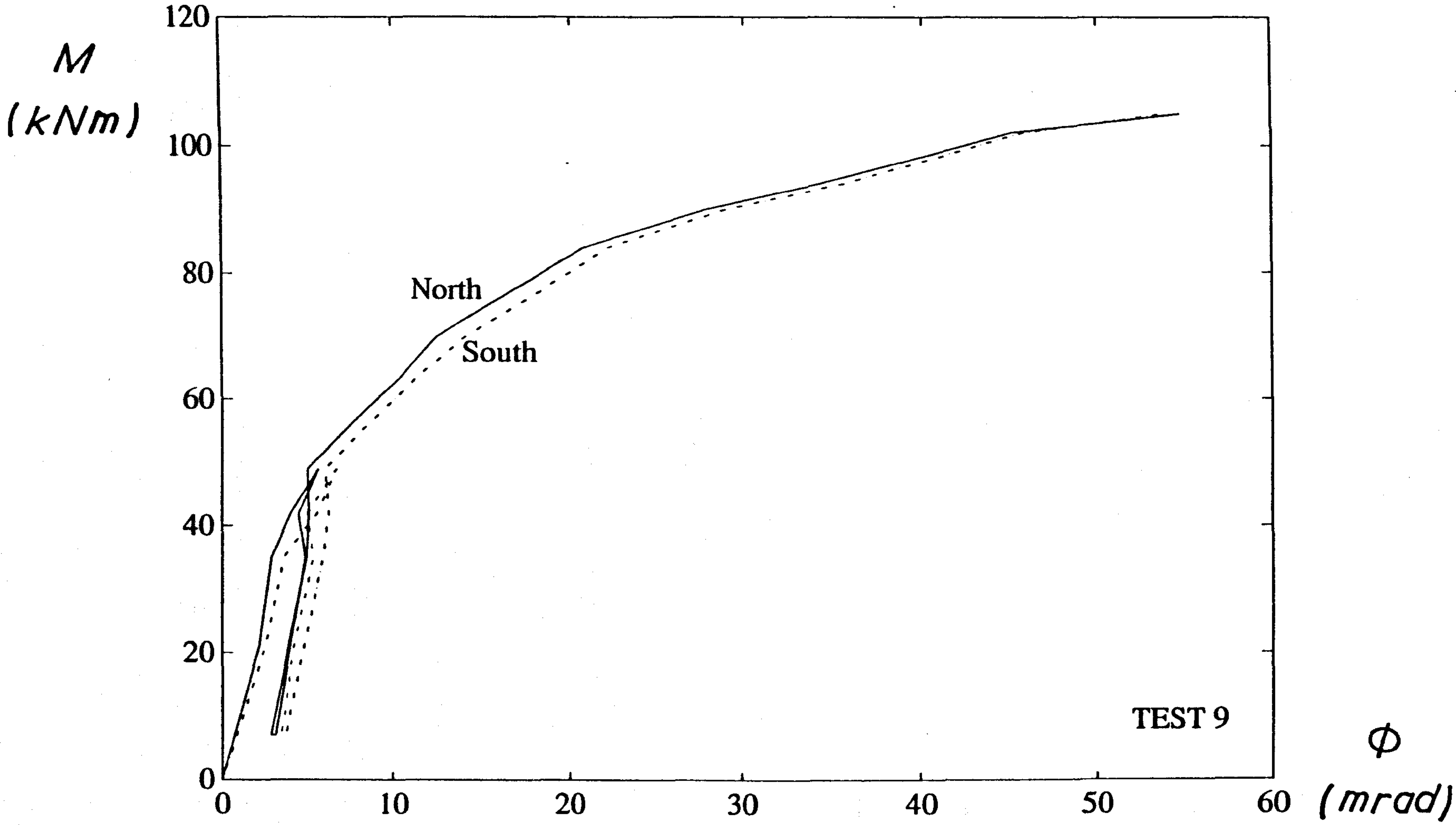


Fig. E-13, Moment-rotation curves measured by manual inclinometer in Test 9

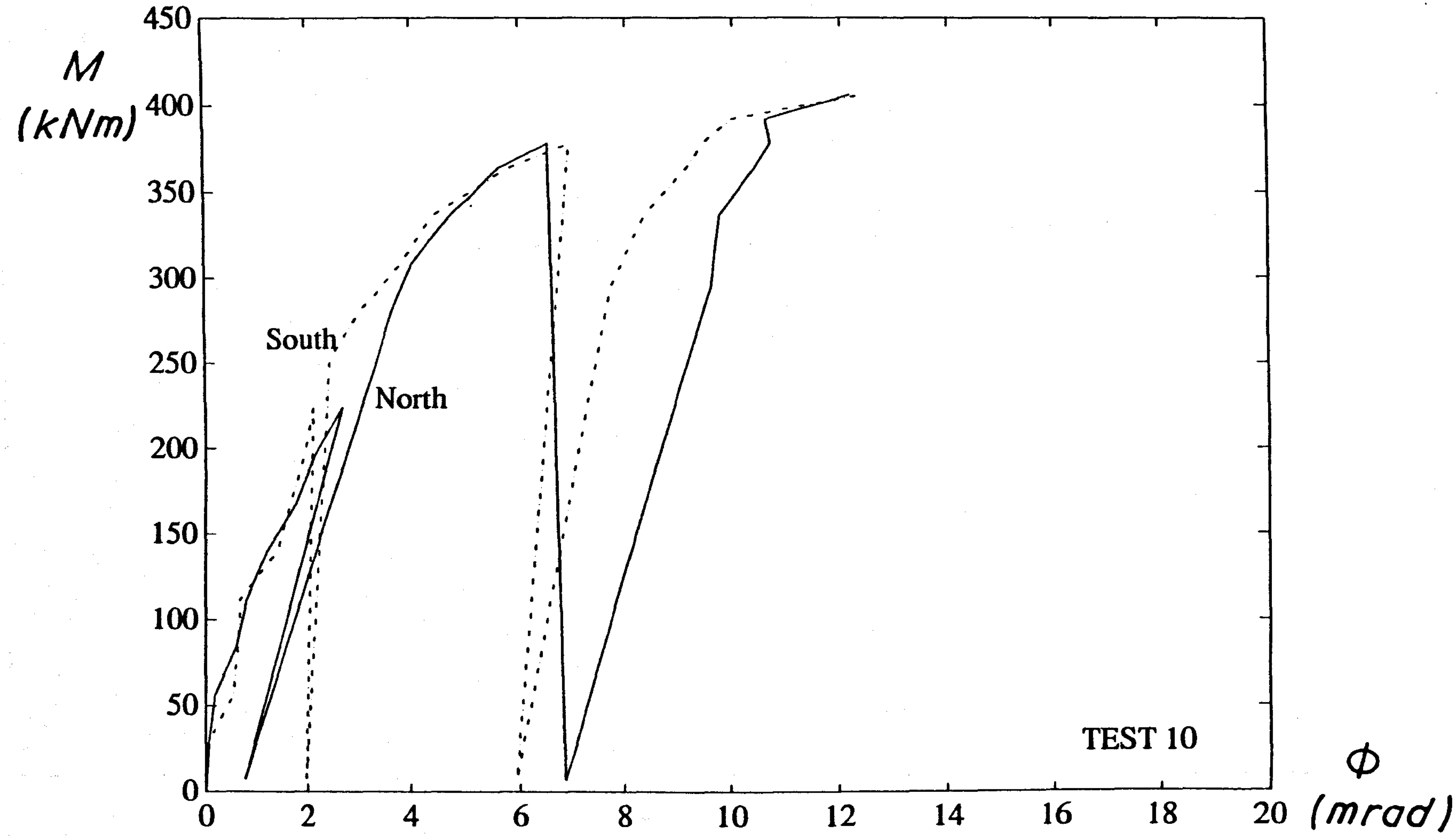


Fig. E-14, Moment-rotation curves measured by manual inclinometer in Test 10

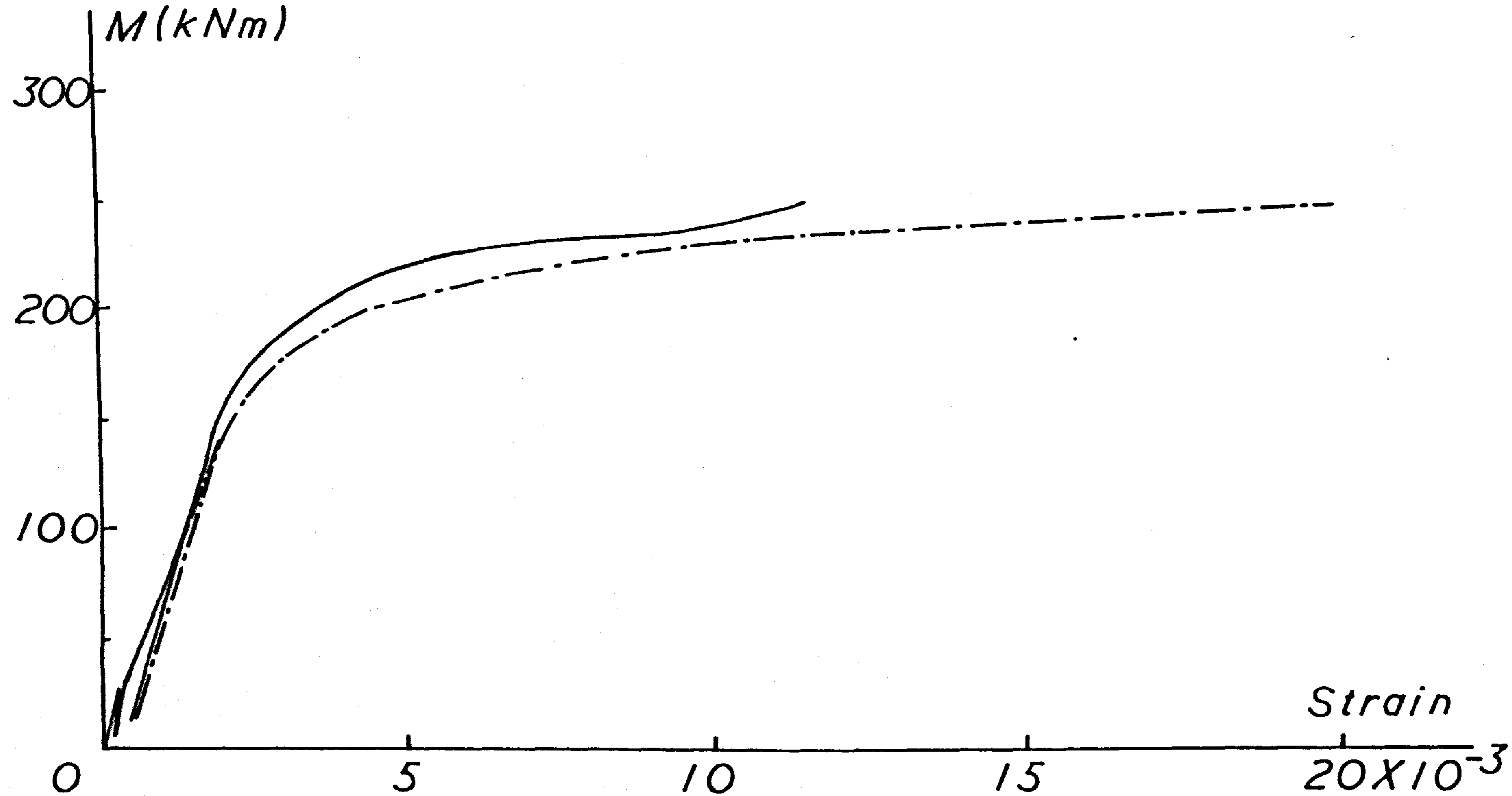


Fig. E-15, Moment-strain curves measured by strain gauges on reinforcement (North side, Test 1)

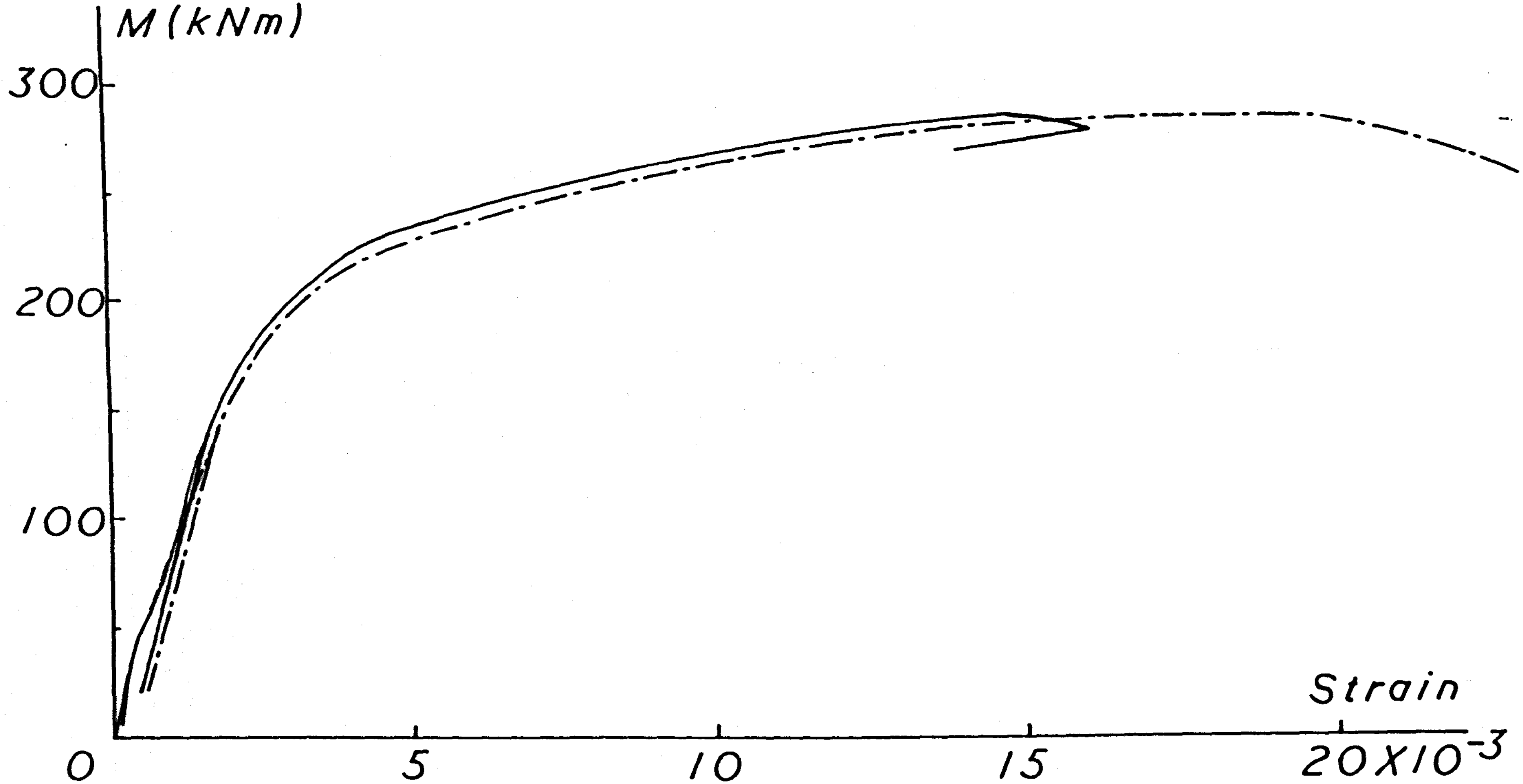


Fig. E-16, Moment-strain curves measured by strain gauges on reinforcement (North side, Test 2)

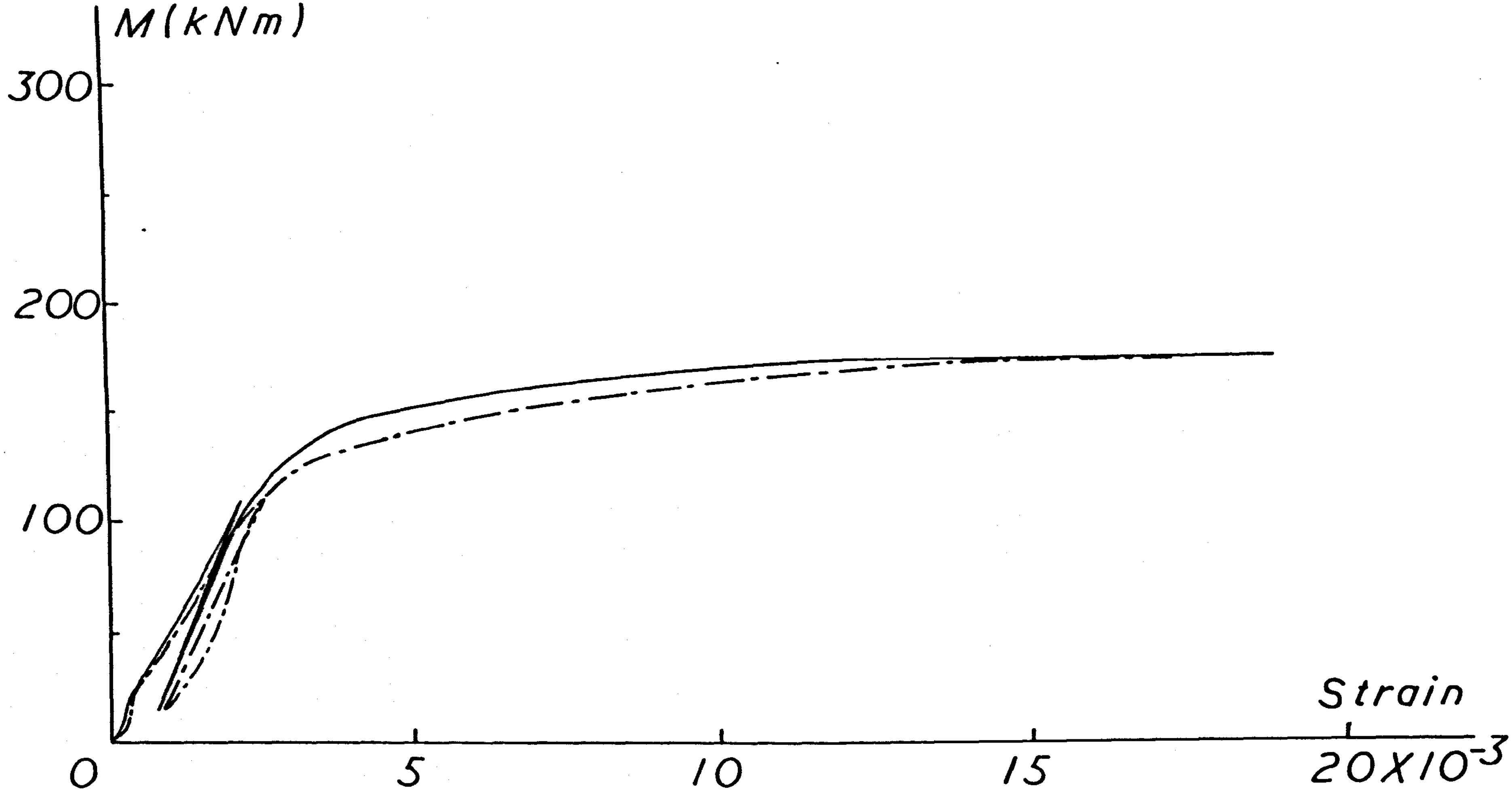


Fig. E-17, Moment-strain curves measured by strain gauges on reinforcement (North side, Test 3)

Appendix F

Crack Widths

Appendix F

CRACK WIDTHS

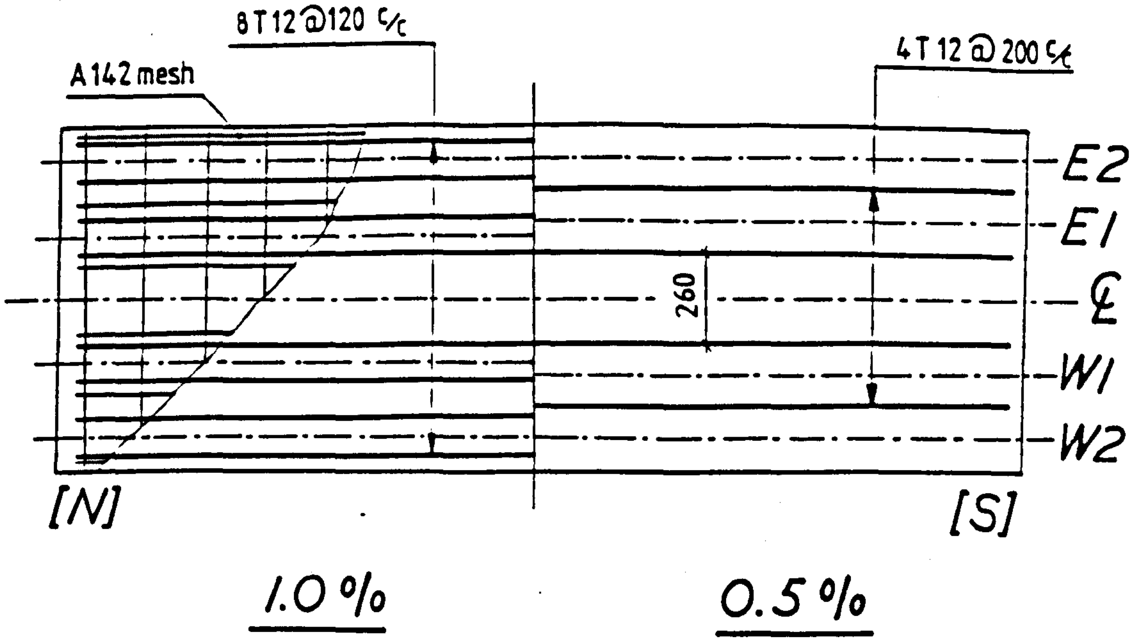
The crack widths were measured in Test 3 at a load level equal to half of the calculated maximum load on the composite connection. They were measured again after the specimen had been unloaded and reloaded to the previous level of loading (half of the calculated maximum load).

In all other composite tests, crack widths were measured at the two-thirds of the calculated maximum load on the connection.

In the minor axis tests, namely Tests 5,6 and 8, crack widths were also measured at loads greater than the two-thirds of the calculated maximum load, to give warning of impending fracture of the reinforcement. This was to be avoided prior to the start of unbalanced loading.

In the tables:

1) The crack widths were measured along the specified lines. These lines are shown in Fig. F-1 as lines E1, E2, Centreline, W1 and W2 (denoting East and West sides).
2) The numbers in the first row of tables belong to the crack patterns (transverse cracks perpendicular to the longitudinal centreline); numbering from the column toward the end of cantilevers.
3) "Dis." means the distance (mm) of a crack being measured with respect to the transverse centreline of the specimen.
4) "Load" means the load level (kN) at which the crack concerned was first observed to have formed.
5) "W" is the width of crack (mm).
6) "N" and "S" denote the north and the south side cantilever respectively.
7) Where no value is given, the crack concerned did not intersect the measurement line mentioned in (1) above.
8) Where more than one value of width is given for a crack pattern at a measurement line, the crack had been divided into several branches intersecting the measurement line.



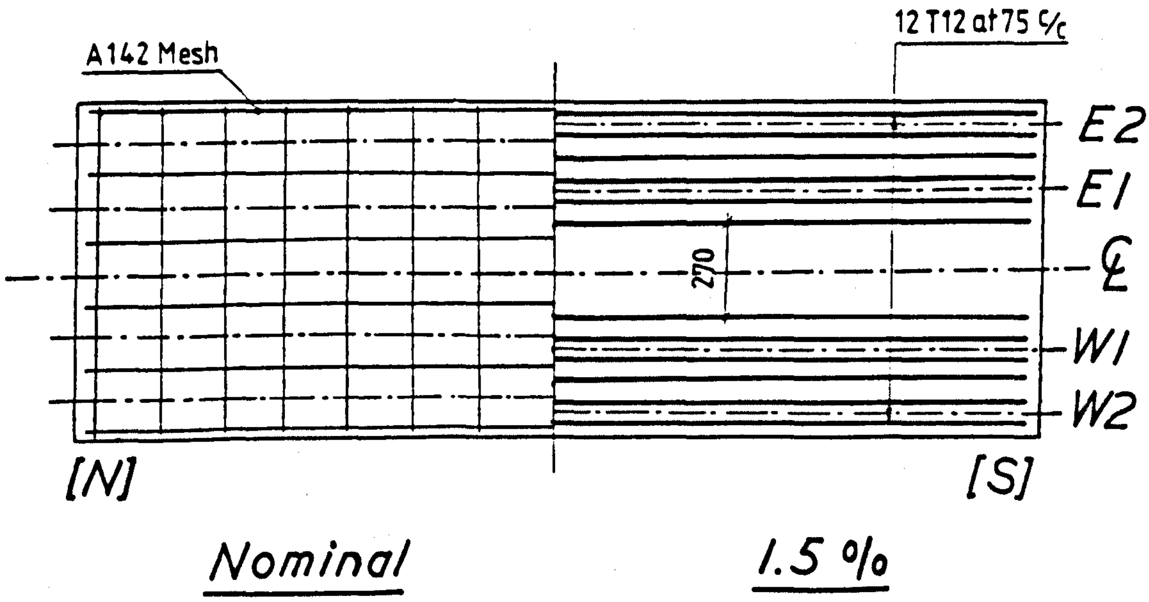


Fig F-1, Lines along which crack widths were measured

Test 3

a BEFORE UNLOADING

b AFTER RELOADING

SIDE			NORTH SIDE					SOUTH SIDE			
CRACK PATTERN			5	4	3	2	1	1	2	3	4
E2	DISTANCE		755	585	395	330	95	135	335	525	820
	LOAD		80	80	80	50	20	30	60	50	80
	WIDTH	a	0.03 / 0.15	0.10	0.03	0.03 / 0.38	0.35	0.05 / 0.20	— / 0.25	0.20	0.10
		b	0.03 / 0.15	0.13	S	0.03 / 0.38	0.33	0.08 / 0.25	S / 0.25	0.23	0.13
E1	DISTANCE		745	605	365	265	130	125	260	545	755
	LOAD		70	80	70	50	20	30	60	50	80
	WIDTH	a	0.18	0.05 / 0.05	0.10 / 0.05	0.18	— / 0.40 / —	0.38	0.13 / 0.05 / 0.13	0.15 / —	0.15
		b	0.18	0.05 / 0.05	0.15 / 0.03	0.20	S / 0.40 / 0.03	0.40	0.18 / 0.05 / 0.13	0.18 / 0.03	0.13
CENTERLINE	DISTANCE		765	615	365	250	100	100	300	495	750
	LOAD		70	80	60	50	30	50	60	50	80
	WIDTH	a	0.05 / 0.05	0.05 / 0.05	0.10	0.23	÷	÷	0.35 / —	0.18	0.10
		b	0.08 / 0.08	0.05 / 0.08	0.13	0.28	÷	÷	0.35 / 0.03	0.20	0.13
W1	DISTANCE		855	600	500	345	120	95	340	560	700
	LOAD		80	80	70	50	20	20	60	50	80
	WIDTH	a	0.13	0.05	— / 0.20	0.25	0.40	0.30 / 0.05 / 0.08	0.40	0.05 / 0.20	0.13
		b	0.15	0.08	S / 0.20	0.30	0.40	0.35 / 0.03 / 0.05	0.43	0.20 / 0.25	0.15
W2	DISTANCE		940	-	565	335	160	80	345	575	750
	LOAD		80	-	70	50	20	20	60	50	80
	WIDTH	a	0.05 / 0.08	—	0.28 / 0.03	— / 0.30	0.40	0.45	0.03 / 0.35 / —	0.30 / —	0.13 / —
		b	0.05 / 0.05	—	0.30 / 0.25	0.03 / 0.25	0.43	0.45	0.03 / 0.35 / 0.03	0.30 / S	0.15 / S

S MEANS TOO SMALL TO MEASURE.

÷ MEANS WAS NOT POSSIBLE TO MEASURE.

Test 4

E or W	SIDE	Pattern:	1	2	3	4	5	6	7	8	9	10	11	12	13
E2	N	DIS.	20	110	180	275 325	405	625	630	730	825	—	1040	—	—
		LOAD	100	100	70	110 70	80	100	90	110	110	—	140	—	—
		W	0.08	0.08	0.10	0.10 0.10	0.13	0.13	0.10	0.13	0.08	—	0.05	—	—
	S	DIS.	35	85	180	230 285	375 430	505 550	635	770	860	950	—	—	—
		LOAD	130	70	70	100 90	70 70	90 130	70	90	110	130	—	—	—
		W	0.05	0.05	0.10	0.08 0.10	0.13 0.05	0.10 0.05	0.05	0.10	0.10	0.03	—	—	—
E1	N	DIS.	15	95	135	290	325	395	535	620	740	910 970	1005	1215	1380
		LOAD	70	100	40	100	60	90	70	110	90	140	130	140	140
		W	0.05	0.05	0.15	0.08	0.15	0.10	0.15	0.05	0.10	0.13 0.03	0.03	0.10	S
	S	DIS.	—	60	135	200	300 365	460	565	680	785	850	1010	1280	—
		LOAD	—	50	20	100	60 70	90	70	120	90	130	120	130	—
		W	—	0.13	0.15	0.08	0.15 0.13	0.13	0.10	0.10	0.10	0.08	0.08	0.05	—
℄	N	DIS.	—	105	260	270	280	—	535	—	755	—	1000	1235	1360
		LOAD	—	÷	50	50	50	—	60	—	80	—	120	130	140
		W	—	÷	0.20	0.20	0.15	—	0.10 0.13	—	0.10 0.08	—	0.15	0.10 0.03	0.05
	S	DIS.	—	—	105	250	—	300	460	—	—	760	975	1205	1335
		LOAD	—	—	÷	50	—	100	70	—	—	80	120	130	140
		W	—	—	÷	0.45	—	0.03	0.13 0.05	—	—	0.20	0.15	0.05 0.13	0.08
W1	N	DIS.	35	95 115	250	315	395	540	555	—	750	835	685	1000	—
		LOAD	120	20	120	50	90	60	60	—	90	140	120	130	—
		W	0.03	0.13 S	0.13 0.05	0.20	0.18	0.03	0.10 0.13	—	0.13	0.08	0.05 0.08	0.05	—
	S	DIS.	7	85	135	285	375	420	535	625	745	790 855	1015	1335	—
		LOAD	50	90	20	50	140	120	80	140	90	140	120	140	—
		W	0.03 0.08	0.13	0.18	0.28	0.05	0.08 0.13	0.10 0.05	0.05	0.10	0.05 0.05	0.05	0.08	—
W2	N	DIS.	35	135	250	320	480	555	625	740	815	930	1015	—	—
		LOAD	120	40	80	60	80	110	80	140	110	140	120	—	—
		W	0.05	0.13	0.10	0.10	0.10	0.10	0.15	0.05	S	0.05	0.05	—	—
	S	DIS.	30	85	145	265 335	395	480	565	735	780	930	1055	—	—
		LOAD	50	120	40	80	130	130	80	140	90	140	130	—	—
		W	0.10	0.03 0.05	0.10	0.10 0.05	0.08	0.05	0.15	0.08	0.08 0.03	0.05	0.03	—	—

Test 5

E or W	SIDE	Pattern:	1	2	3	4	5	6	7
E2	N	DIS.	95	250	420	—	655	730	940
		LOAD	80	40	70	—	60	110	120
		W	0.18	0.10	0.10	—	0.05	0.05	0.05
	S	DIS.	15	140	195	260	440	615	950
		LOAD	60	60	80	90	60	90	110
		W	0.08	0.05	0.05	0.05	0.08	0.08	0.08
E1	N	DIS.	65	215	410	—	660	—	885
		LOAD	80	40	70	—	60	—	120
		W	0.10	0.10	0.08	—	0.08	—	0.03
	S	DIS.	30	—	120	260	365	620	905
		LOAD	80	—	40	90	60	90	110
		W	0.05	—	0.13	0.05	0.13	0.10	0.05
℄	N	DIS.	—	200	400	—	655	710	880
		LOAD	—	40	70	—	60	90	120
		W	—	0.18	0.08	—	0.03	0.05	0.05
	S	DIS.	—	75	140	—	400	670	800
		LOAD	—	80	110	—	60	90	100
		W	—	0.03	0.05	—	0.15	0.08	0.03
W1	N	DIS.	80	185	375	490	540	650	790 | 900
		LOAD	50	40	70	100	70	90	120 | 120
		W	0.10	0.10	0.05	0.05	S	0.05	S | 0.05
	S	DIS.	15	75	165	420	—	645	875
		LOAD	40	70	110	60	—	80	100
		W	0.05	0.13	S	0.13	—	0.10	0.05
W2	N	DIS.	60	235	370	485	540	690	900
		LOAD	60	40	80	100	80	90	120
		W	0.05	0.10	0.05	0.05	0.05	0.13	0.03
	S	DIS.	55	160	260	395	510	620 | 710	875
		LOAD	40	70	90	60	80	80 | 120	100
		W	0.05	0.08	0.05	0.08	0.03	0.05 | 0.05	0.05

Test 6			at 35 kN			at 55 kN			at 83 kN		
E or W	SIDE	Pattern:	1	2	3	1	2	3	1	2	3
E2	N	DIS.	200			200			200		675
		LOAD	30			30			30		55′
		W	0.15 | 0.10			0.45 | 0.10			0.83 | 0.03		0.13 | 0.13
	S	DIS.	25			25	250		25	250	875
		LOAD	25			25	40		25	40	70
		W	0.05			0.08	0.30		0.35 | 0.20	0.63	0.25
E1	N	DIS.	150			150			150		645
		LOAD	30			30			30		55′
		W	0.25			0.35 | 0.20			0.80 | 0.15		0.25 | 0.03
	S	DIS.	5	200		5	200		5	200	880
		LOAD	25	35		25	35		25	35	70
		W	0.05	0.03		0.08	0.35		0.40 | 0.10	0.60	0.23
℄	N	DIS.					210			210	650
		LOAD					40			40	55′
		W					0.25			0.53	0.28
	S	DIS.		210			210			210	865
		LOAD		30			30			30	70
		W		0.05			0.13 | 0.08			0.38 | 0.13	0.20
W1	N	DIS.					205			205	660
		LOAD					40			40	55′
		W					0.35			0.95	0.25 | 0.05
	S	DIS.	170	225		170	225		170	225	855
		LOAD	30	25		30	25		30	25	70
		W	0.13	0.13		0.13 | 0.03	0.33		0.25 | 0.08 | 0.08 | 0.05	0.60	0.18
W2	N	DIS.					250		205	250	645
		LOAD					40		83	40	55′
		W					0.38		0.30	0.65	0.30
	S	DIS.		235			235			235	870
		LOAD		25			25			25	70
		W		0.13 | 0.20			0.20 | 0.35			0.85 | 0.13	0.15

Test 7

E or W	SIDE	Pattern:	1	2	3	4	5	6	7	8	9	10	11
E2	N	DIS.	70 / 100	180	—	300	360	505	555 / 620	710	760 / 810	910	1010
		LOAD	130 / 50	80	—	60	60	110	70 / 120	100	130 / 120	110	120
		W	0.03 / 0.05	0.05	—	0.03	0.03	0.03	0.1 / 0.05	0.05	0.05 / 0.03	0.05	0.05
	S	DIS.	15	100	—	225	320	390	520	640	745	830	1050
		LOAD	70	40	—	60	70	60	90	90	100	110	120
		W	0.10	0.10	—	0.05	0.05	0.10	0.10	0.08	0.05	0.05	0.03
E1	N	DIS.	60	130	240	—	340	450	525	600	780	820	995
		LOAD	40	30	50	—	60	90	70	120	80	110	120
		W	0.03	0.10	0.10	—	0.08	0.05	0.10	0.05	0.08	0.05	0.05 / 0.05
	S	DIS.	10	85	170	190	—	300	450	555	760	775	990
		LOAD	70	70	40	60	—	60	90	90	120	90	120
		W	0.08	0.05	0.05	0.05	—	0.10	0.08	0.05	0.03	0.05	0.05
℄	N	DIS.	—	—	110	—	—	220	440	—	610	—	900
		LOAD	—	—	130	—	—	60	70	—	70	—	120
		W	—	—	0.10	—	—	0.20	0.05	—	0.10	—	0.08
	S	DIS.	—	—	—	140	270	—	380	490	—	710	990
		LOAD	—	—	—	110	60	—	90	90	—	70	120
		W	—	—	—	0.05	0.15	—	0.05	0.10	—	0.10	0.05
W1	N	DIS.	45	125	170	260	305	380	480	575	745	—	1005
		LOAD	60	30	110	90	60	70	80	80	90	—	120
		W	0.13	0.05	0.05	0.05	0.10	0.13	0.05	0.10	0.05	—	0.05
	S	DIS.	30	85	140	260	290	365	470	570	—	770	980 / 1035
		LOAD	110	50	30	60	100	70	90	100	—	90	130
		W	0.05	0.1	0.05	0.05	0.05	0.08	0.05	0.05	—	0.05	0.05 / 0.05
W2	N	DIS.	30	90	—	250	290	360 / 385	520	595	750	—	975
		LOAD	110	30	—	50	100	60 / 80	80	70	90	—	120
		W	0.05	0.10	—	0.05	0.05	0.05 / 0.08	0.10	0.05	0.05	—	0.05
	S	DIS.	65	60	—	230	—	350	505	590	750	825	990
		LOAD	70	50	—	60	—	70	90	100	100	130	130
		W	0.10	0.05	—	0.05	—	0.10	0.08	0.10	0.05	0.03	0.05

Test 8

E or W	SIDE	Pattern:	1	2	3	4	5	6
E2	N	DIS.	25	185	265	485		900
		LOAD	30	70	100	60		90
		Ws	0.25	0.15	0.05	0.08 / 0.10		0.05
		Wu	0.30	0.05 / 0.35	0.10 / 0.15	0.05 / 0.25		0.10
	S	DIS.		200	370		530	
		LOAD		50	60		90	
		Ws		0.25	0.03 / 0.10		0.10	
		Wu		0.35	0.05 / 0.25		0.20	
E1	N	DIS.	10	200	275	490		905
		LOAD	30	70	100	60		90
		Ws	0.15 / 0.08	0.05 / 0.10	0.05	0.05 / 0.08		0.05
		Wu	0.10 / 0.05	0.05 / 0.35	0.10	0.05 / 0.20		0.10
	S	DIS.		175	355		545	
		LOAD		50	60		90	
		Ws		0.20	0.15		0.10	
		Wu		0.65	0.30		0.20	
℄	N	DIS.		205		425	505	850
		LOAD		70		60	80	90
		Ws		0.15		0.05	0.08	0.05
		Wu		0.05 / 0.33		0.20	0.20	0.10
	S	DIS.		275	315	490	555	
		LOAD		80	60	100	90	
		Ws		0.08	0.13	0.03	0.15	
		Wu		0.08	0.20	0.05	0.15	
W1	N	DIS.	100	260		420	490	855
		LOAD	40	70		60	80	90
		Ws	0.15	0.18		0.18	0.05	0.08
		Wu	0.10 / 0.65	0.30		0.25	0.10	0.10
	S	DIS.	50	265	325	485	645	
		LOAD	30	70	60	90	90	
		Ws	0.25	0.15	0.13	0.05	0.13	
		Wu	0.48	0.08 / 0.30	0.08	0.05 / 0.08	0.20	
W2	N	DIS.	110	260		475		950
		LOAD	60	70		60		90
		Ws	0.13	0.18		0.25		0.05
		Wu	0.60	0.05 / 0.35		0.38		0.30
	S	DIS.	0	265	350	470	670	
		LOAD	30	70	60	90	90	
		Ws	0.25	0.10	0.18	0.15	0.15	
		Wu	0.60	0.30	0.25	0.20	0.20	

W_s : Crack widths at SLS (calculated) Load = 100kN

W_u : " " " ULS (") Load = 145kN

Test 10

E or W	SIDE	Pattern:	1	2	3	4	5	6	7
E2	N	DIS.	—	145	—	275 / 310	470	580	735
		LOAD	—	70	—	100 / 130	110	130	130
		W	—	0.13	—	0.15 / 0.03	0.03	0.05	0.05
	S	DIS.	15	110	185	330	485 / 510	750	865
		LOAD	110	80	100	170	130 / 130	170	130
		W	0.10	0.08	0.10	0.20	0.10 / 0.03	0.05	0.05
E1	N	DIS.	45	145	260	315	510	—	745
		LOAD	100	60	130	100	120	—	130
		W	0.13	0.10	0.05	0.15	0.10	—	0.10
	S	DIS.	—	70 / 110	180	325	500	—	810
		LOAD	—	60 / 60	140	90	130	—	110
		W	—	0.05 / 0.15	0.05	0.18	0.10	—	0.10
℄	N	DIS.	—	—	—	305	—	500	715
		LOAD	—	—	—	100	—	100	130
		W	—	—	—	0.25	—	0.15	0.05
	S	DIS.	—	—	220	325	500	—	750
		LOAD	—	—	160	90	130	—	110
		W	—	—	0.05	0.20	0.10	—	0.08
W1	N	DIS.	15	120	200	385	490	575	725 / 785
		LOAD	130	60	90	100	100	120	130 / 170
		W	0.05	0.10	0.13	0.15	0.05	0.08	0.05 / 0.05
	S	DIS.	—	85	230	330	525	740	780
		LOAD	—	60	120	90	130	170	110
		W	—	0.15	0.10	0.15	0.10	0.03	0.05
W2	N	DIS.	—	110 / 190	305	435	555	660	800
		LOAD	—	80 / 40	90	110	100	150	130
		W	—	0.05 / 0.05	0.10	0.05	0.10	0.05	0.08
	S	DIS.	0	85	195	335	495 / 600	730	850
		LOAD	150	60	120	90	130 / 140	140	150
		W	0.0	0.10	0.10	0.20	0.10 / 0.05	0.05	0.08

Appendix G

Derivation of Strain Relationships

Appendix G

DERIVATION OF STRAIN RELATIONSHIPS

From Fig. G-1, the following expression can be written for strain at any depth of the section in propped construction:

$$\varepsilon_x = -\psi\,\varepsilon_y\,(\frac{D'-x}{D'}) \qquad \text{(G.1)}$$

To find the depth at which strain equals the value of yield strain, ε_y is substituted for ε_x, which results in:

$$x = (\frac{\psi-1}{\psi})\,D' \qquad \text{(G.2)}$$

For a given value of ψ and $x = D$, from Eqn. (G.1):

$$\frac{f_{top}}{f_y} = -\psi\,\varepsilon_y\,(1-\frac{D}{D'}) \qquad \text{(G.3)}$$

Since $E\varepsilon_y = f_y$, therefore:

$$\frac{f_{top}}{f_y} = -\psi\,(1-\frac{D}{D'}) \qquad \text{(G.4)}$$

The moment capacity of composite section in sagging bending is given by Gibbons(1992) as the following equation, taking moments about the centre of compression (see Fig. G-2):

$$M_{ps} = A_w f_y D\,[\frac{A_f}{A_w}\,(1+\frac{D_c}{D}) + x\,(1-\frac{x}{2}+\frac{D_c}{D}) + (\frac{x_0-x}{2})\,(\frac{2}{3}(x_0-x) + (1-x_0) + \frac{D_c}{D})$$

$$-(\frac{1-x_0}{2})\,\frac{f_{top}}{f_y}\,(\frac{D_c}{D} + \frac{1-x_0}{3}) - \frac{A_f}{A_w}\,\frac{f_{top}}{f_y}\,\frac{D_c}{D}] \qquad \text{(G.5)}$$

For situations where f_{top} is tensile, the last two terms of Eqn. (G.5) will be replaced by:

$$A_w f_y D\,[(\frac{f_{top}}{2f_y}+\frac{1}{2})\,(1-x)\,(\frac{D_c}{D} + \frac{1-x}{2}) + \frac{A_f}{A_w}\,\frac{f_{top}}{f_y}\,\frac{D_c}{D}]$$

The values of x and x_0 in Eqn. (G.5) are non-dimensional taken as the ratio of x and x_0 in Fig. G-2 to the beam depth D.

The proportions of moment capacity achieved in the section for the cases of Tests 1 and 10 are tabulated in Table G-1. From the first and the last columns of the table, it is deduced that 0.95 M_{ps} corresponds approximately to values of $4.5\varepsilon_y$ for propped and $6.0\varepsilon_y$ for unpropped beam.

From Eqn. (8.13), assuming $s = 1.3$, the yielded length l_y will be:

$$l_y = 0.24\,L\,(1+k)^{-1/2} \tag{G.6}$$

Substituting l_y from Eqn. (G.6) in Eqn. (8.14) and assuming $\frac{D}{D'} = 0.91$, the plastic rotation can be written as:

propped beam $\varepsilon_{midspan} = 4.5\varepsilon_y$ $\theta_p = 0.38\,\frac{f_y L}{ED}\,(1+k)^{-1/2}$

unpropped beam $\varepsilon_{midspan} = 6.0\varepsilon_y$ $\theta_p = 0.55\,\frac{f_y L}{ED}\,(1+k)^{-1/2}$

Table G-1

Type of beam	ψ	$\frac{D_s}{D}$	$\frac{D}{D'}$	$\frac{x}{D}$	$\frac{f_{top}}{f_y}$	$\frac{D_c}{D}$	$\frac{A_f}{A_w}$	$\frac{M}{M_{ps}}$
propped	3	0.40	0.86	0.78	0.42	0.28	0.38	0.93
	4	0.40	0.86	0.87	0.56	0.28	0.38	0.95
	5	0.40	0.86	0.93	0.70	0.28	0.38	0.97
	3	0.25	0.91	0.73	0.27	0.18	1.00	0.92
	4	0.25	0.91	0.82	0.36	0.18	1.00	0.94
	5	0.25	0.91	0.88	0.45	0.18	1.00	0.95
unpropped	3	0.40	0.86	0.72	0.09	0.28	0.38	0.88
	4	0.40	0.86	0.81	0.22	0.28	0.38	0.91
	5	0.40	0.86	0.87	0.37	0.28	0.38	0.95
	4	0.25	0.91	0.77	0.03	0.18	0.38	0.91
	5	0.25	0.91	0.83	0.12	0.18	0.38	0.93
	5	0.25	0.91	0.83	0.12	0.18	1.00	0.91
	6	0.25	0.91	0.87	0.21	0.18	1.00	0.93

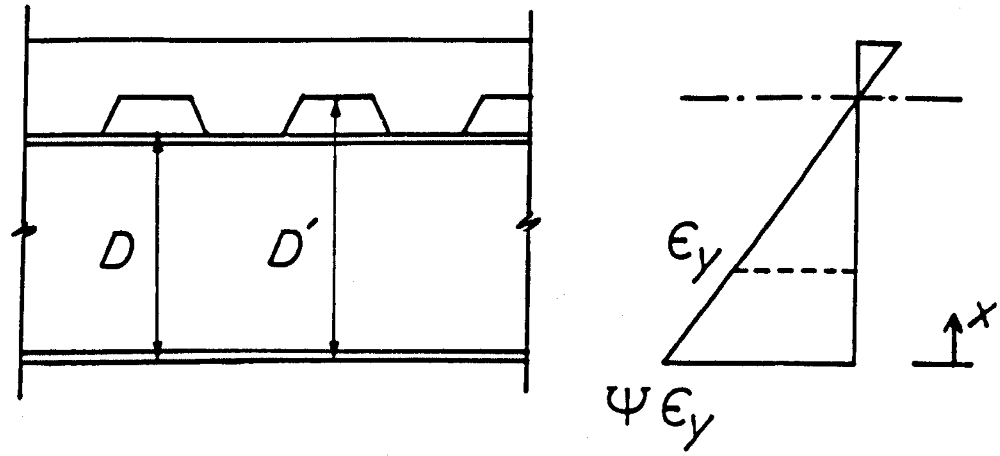


Fig. G-1, Strain block

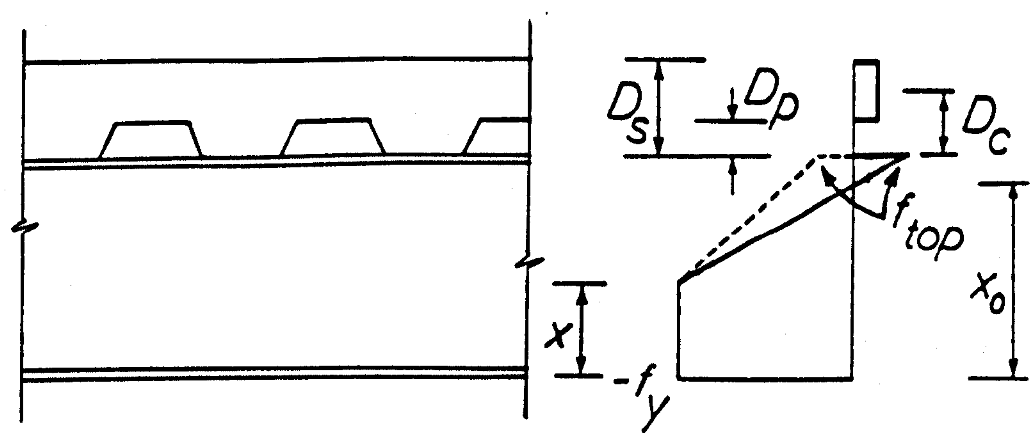


Fig. G-2, Stress block

Appendix H

Sample Check for Required Rotation

Section	Project Supervisor

Remarks

The nearest steel section which has a second moment of area similar to that of composite uncracked section is:

406×178 UB 67 , $I = 24300\ cm^4$

Fig. 1-9 or Fig. 8-7

Beam line: $M - \frac{wL^2}{12} = \frac{wL^2/12}{wL^3/24EI}\,\phi$

$\therefore M = \frac{wL^2}{12} - \frac{2EI}{L}\phi$

$\therefore W = \frac{12}{L^2}\left(M + \frac{2EI}{L}\phi\right)$

① For $\begin{cases} M = 258 \text{ kNm} \\ \phi_1 = 14.1 \text{ mrad} \end{cases}$

$W_1 = \frac{12}{9^2}\left(258 + \frac{2 \times 210 \times 10^6 \times 24300 \times 10^{-8}}{9} \times 14.1 \times 10^{-3}\right)$

$= 61.9\ kN/m$

$\frac{w_1 L^3}{24EI} = \frac{61.9 \times 9^3 \times 10^6 \times 10^{-3}}{24 \times 210 \times 24300 \times 10^4} = 36.8$ mrad

This intersects at $M = 258$, $\phi = 14.1$ see Fig. H-1.

$M_{midspan} = \frac{wL^2}{8} - 258 = \frac{61.9 \times 9^2}{8} - 258$

$= 369$ kNm

Design guide SCI

$M_{Pl.Rd} = 371$ kNm Grade 43

$= 479$ kNm Grade 50

Section	Project Supervisor

Remarks

Adopt Grade 50

$\therefore 479 + 258 = \frac{W_2 L^2}{8}$

$W_2 = \frac{(479 + 258) \times 8}{9^2} = 72.8\ kN/m$

$\Delta W = W_2 - W_1 = 72.8 - 61.9 = 10.9\ kN/m$

$\Delta\phi = \frac{\Delta W . L^3}{24EI} = \frac{(10.9) \times 9^3 \times 10^8}{24 \times 210 \times 24300 \times 10^6} \times 10^3$

$= 6.5$ mrad

$\phi_2 = 14.1 + 6.5 = 20.6$ mrad

② For $\begin{cases} M = 258 \text{ kNm} \\ \phi_1 = 16.9 \text{ mrad} \end{cases}$

$w_1 = \frac{12}{9^2}\left(258 + \frac{2 \times 210 \times 10^6 \times 24300 \times 10^{-8}}{9} \times 16.9 \times 10^{-3}\right)$

$= 66.6\ kN/m$

$\Delta W = W_2 - W_1 = 72.8 - 66.6 = 6.2\ kN/m$

$\Delta\phi = \frac{\Delta W . L^3}{24EI} = \frac{(6.21) \times 9^3 \times 10^8}{24 \times 210 \times 24300 \times 10^6} \times 10^3$

$= 3.7$ mrad

$\phi_2 = 16.9 + 3.7 = 20.6$ mrad

$\therefore$ Same rotation irrespective of the $M - \phi$ relation (see Fig. H-1).

H-1

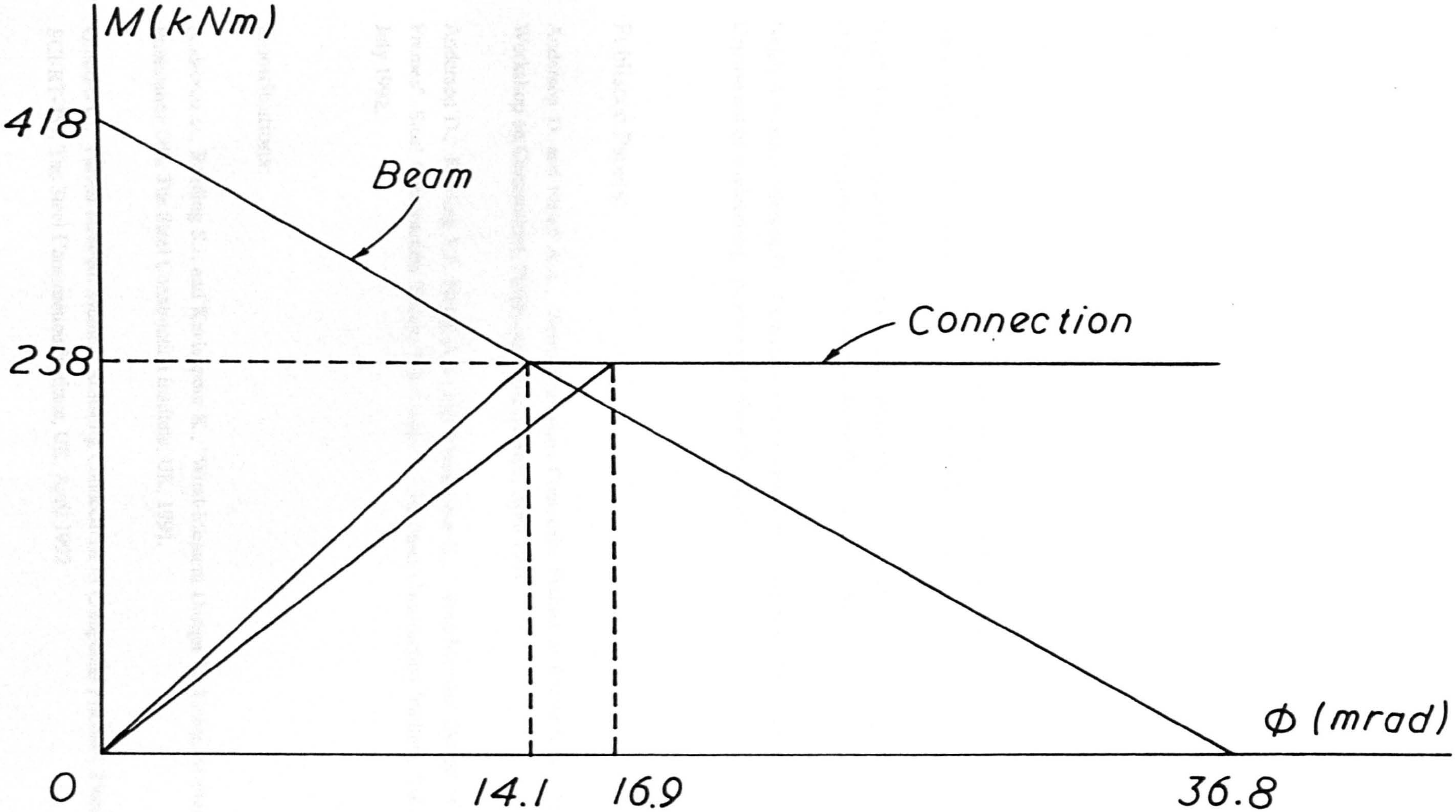


Fig. H-1, Sample check on required rotation using beam line method

Publications

1) Technical Reports, presented in the meetings at the Building Research Establishment, on the experimental programme authorised by the Steel Construction Institute:

Anderson D. and Najafi A.A., "Composite connections with structural steel end plates - a preliminary report", Department of Engineering, University of Warwick, December 1990.

Najafi A.A. and Anderson D., "Composite connections with structural steel end plates", second report, Department of Engineering, University of Warwick, July 1991.

Najafi A.A. and Anderson D., "Composite connections with structural steel end plates", third report, Department of Engineering, University of Warwick, December 1991.

Najafi A.A. and Anderson D., "Composite connections with structural steel end plates", fourth report, Department of Engineering, University of Warwick, April 1992.

2) Published Papers:

Anderson D. and Najafi A.A., "Semi-continuous Composite Frames in Eurocode 4", AISC/ECCS Workshop on Connections, Pittsburgh, Pennsylvania, April 1991.

Anderson D., Reading S.J., Najafi A.A. and Kavianpour K., "Wind-Moment Design of Unbraced Frames", Steel Construction Today, The Journal of the Steel Construction Institute, Vol. 6, No. 4, July 1992.

3) Contributions:

Anderson D., Reading S.J. and Kavianpour K., "Wind-Moment Design for Unbraced Frames", SCI Publication 082, The Steel Construction Institute, UK, 1991.

Gibbons C., "Partial Strength Moment Resisting Connections in Composite Frames", Document No. SCI-RT-257, The Steel Construction Institute, UK, April 1992